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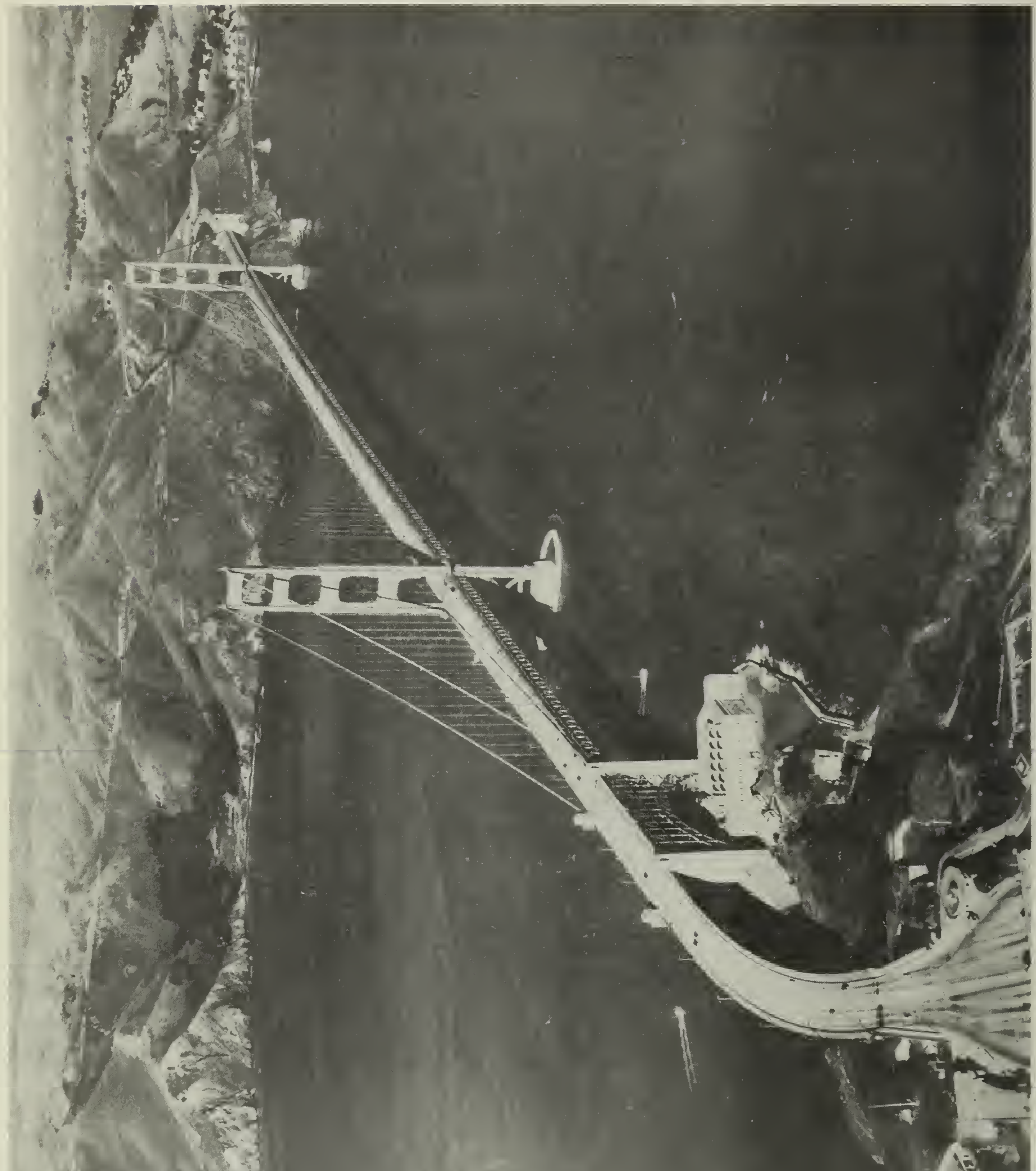
THE GOLDEN GATE BRIDGE

REPORT OF THE GENERAL MANAGER

GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

A SYMPOSIUM

MAY 1958



The Golden Gate Bridge Looking North from San Francisco
Pacific Ocean Is to the Left and Historic Fort Point in Foreground

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San Francisco, California

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Counties

San Francisco
Marin
Mendocino
Sonoma
Napa
Del Norte

GOLDEN GATE BRIDGE
AND HIGHWAY DISTRICT
BOX 99, PRESIDIO STATION • FILLMORE 6-5858 • SAN FRANCISCO, CALIFORNIA

May 1, 1958

To The Honorable Board of Directors
Golden Gate Bridge and Highway District

Gentlemen:

Transmitted herewith is a report containing a symposium of various documents that have been prepared over the period of years since opening of the Golden Gate Bridge to traffic in 1937.

These works cover elements of design and construction of the bridge, subsequent structural improvements and data relating to the factor of structural stability, particularly as related to important aspects of geology, seismology and aerodynamics.

Respectfully,

James Adam
James Adam
General Manager
Golden Gate Bridge and
Highway District

A SYMPOSIUM ON STRUCTURAL DESIGN AND STABILITY OF THE GOLDEN GATE BRIDGE

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PART ONE

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A SYMPOSIUM ON STRUCTURAL DESIGN AND STABILITY OF THE GOLDEN GATE BRIDGE

PART ONE

ANALYSIS OF PAST AND PRESENT STATUS

A - GENERAL

Introduction

This symposium has been compiled for a twofold purpose. First, it affords a condensed historical treatise, briefly bridging the gap between early stages of design and construction of the bridge and its present status. Secondly, it serves as a means of assembling the work of noted engineers and scientists as related to the physical stability of the bridge, which in turn has a direct bearing upon those considerations that should be controlling in establishing multi-risk insurance rates and coverage.

Brief Historical Comment

History of the Golden Gate Bridge covers a span of about forty years, going back to 1918 when the first specific action was taken to bring into realization a dream that had long existed, of bridging the world famous Golden Gate at the entrance to the equally renowned San Francisco harbor.

A detailed account of the historic events transpiring during that period and a listing of the many pioneers whose persistent courage brought the bridge into being, would justify many volumes on the subject.

It is not expedient at this time, however, to compile such an extensive historical chronology, and therefore this report has been confined to assembling a number of important individual reports in the form of a symposium, that will conclusively establish the stability characteristics of the Bridge.

Long Range Planning Program

There appears to be need for such a work that will bring together the highlights of background material, relating to those aspects of the Golden Gate Bridge and Highway District that are important to a systematic program of long range planning. Such a program is presently under way, embracing all phases of the District's functional and financial status, and this subject of insurance is an important part thereof.

Fundamental Objectives

Fundamentally, the District, as a legal and physical entity, consists essentially of an administrative cloak tailored by statute, that is draped about the phenominal structure of the Golden Gate Bridge, to keep it groomed physically, adequately provided for financially, and functionally capable of meeting its traffic obligation with passage of time.

Cost of the structure in 1937 was $35,000,000. Today, 21 years later, the cost to reproduce it is estimated at $111,373,000, exclusive of interest during construction and engineering costs.

With a remaining obligation as of June 30, 1958, of $34,663,800 to cover bond interest and redemption still outstanding, it is imperative that all prudent measures be taken by management to protect the presently favorable financial stability of the District, by whatever means may be available.

Sources of Security

Financial security can be attained through two primary approaches. One is to accummulate a reasonable reserve fund from earnings, and the other is to provide insurance coverage in amount commensurate with present day replacement cost and the degree of risk involved.

Prudent and farsighted management has ably maintained a reasonable balance of reserve funds and, in addition, maximum multi-risk insurance coverage within the availability limits of dependable underwriting agencies.

Insurance Coverage and Risk

Insurance premium rates and coverage are, of course, related to the estimated risk and the degree of risk is logically a function of experience. On a structure such as the Golden Gate Bridge, which exceeded precedent in dimensional magnitude, the experience factor was of limited scope. Consequently, it was only natural that the subject of insurance would be approached with a greater degree of caution than might have been the case if a greater scope of experience had existed.

Since 1937 when the Golden Gate Bridge was opened to traffic, there have been many new bridges built that have gradually encroached upon the envious record of this great structure. It still maintains supremacy in total length of a single suspended span, but even that record is now being challenged. Experience gained from this pioneering project has laid the foundation for confidence in design of other long span structures, some having already been built in the east, and others in process of design.

Favorable Experience Record

The Golden Gate Bridge has created its own experience record, the validity of which is attested to by the fact that other super-span structures have been subsequently designed and built with full confidence. Some of those newer structures have already added their experience to that of the Golden Gate Bridge, which results in an increasing foundation of ever widening scope in long span bridge history.

This great bridge has magnificently stood the test of time, through storms of adversity in initial planning and financing, the forces of nature through wind and gales off the Pacific, battering of ocean waves and currents, earthquake forces, economic depression and world conflict, none of which have left a lasting scar. These are momentous achievements that should be looked upon with serious contemplation when considering the probability of future durability.

Conservative Design Factors

In retrospect it can be seen that virtually by force of necessity, the design factors of this pioneer of great bridges were proportioned with over-abundant conservatism. This was an unprecedented undertaking. Its planning, design and construction was guided by equally great men in the field of science and engineering, whose professional reputations were at stake.

In running the gamut of organized opposition, which in large measure appeared to have been fostered by selfish financial motives, the affairs of the District and specifications of the bridge were subjected to close scrutiny of not only the administrative governments of the several counties involved, but also to close examination by the well-qualified engineers on their staffs, outside engineering and scientific consultants, the State Department of Public Works, the State Legislature, the State Supreme Court, the Federal District Court, the U. S. Coast and Geodetic Survey, U. S. Army, U. S. Navy, U. S. War Department, U. S. Department of Commerce, and last but not least, the financial scrutiny of what was then the forerunner of today's greatest banking and financial institution in the world.

It should be obvious that with this array of scientific, governmental and financial exploratory agencies involved, all of which had important interests at stake, had there been serious flaws in design, foundation or construction, they could not have been screened from public view. To the contrary, there is every evidence that the structure is based upon solid foundation, is designed with far greater than necessary factors of safety as to stresses and strength of materials, and more than ample provision for meeting the forces of nature, whether they be elements of weather, geologic, seismologic or aerodynamic disturbance.

Interim Structural Improvements

During intervening years, the Board of Directors of the District has perpetuated that spirit of conservatism of the bridge designers by employing all reasonable means of safeguarding the structure. Wherever experience has indicated a reasonable need for structural or operational improvement, the Board has authorized the necessary funds to accomplish the objective. This should not be construed as picking up deficiencies of original design. Original design was abundantly ample and stress allowances were on the side of conservatism from the beginning. These subsequent improvements were for the purpose of increasing that margin of safety and reducing the potential cost of normal bridge maintenance that might otherwise result.

Purpose of Symposium

This symposium has been compiled to point out the imposing array of evidence in substantiation of the contended stability of this bridge. It is hoped that the result will be to engender a greater degree of confidence in the minds of outside agencies concerned with the financial and physical aspects of the structure.

To those of the bridge administrative group who are so closely associated with its operation and construction, there is no shade of question. Likewise, to the counties of the District and the drivers and occupants of the 15 million vehicles using the bridge annually, there is no question as to its safety, permanence and capabilities.

There is every reason to expect that this confidence should also be reflected in the process of measuring the reasonable extent of insurable risk, especially when such confidence can be so well justified by the sound record of experience that has been amplified by each passing year.

Purchasing power of the dollar is a vitally important factor in this process of insuring structures. Cost to reproduce the bridge has increased by 4. 5 times. Therefore, what would have been a reasonable coverage on original investment should today be likewise increased by 4. 5 times to maintain a proper balance. Insurance coverage and premium rates should consistently be expected to fluctuate with the economic trend. Such coverage is short-term in comparison with a life insurance policy which carries its original rate and coverage to death of the insured. Where periodic adjustments are standard practice in structure insurance, both the rates and the coverage should reflect not only the economic trend, but also changes in proof of risk based on actual experience for which there is no substitute.

B - RESUME OF FINDINGS

General

In the following four parts of this report will be found a condensed assemblage of data that has direct bearing upon the merits of the foregoing discussion.

Part Two deals with certain technical aspects of bridge planning, design and construction.

Part Three contains reproduced documents relating to the structural improvements and studies thereon, that have been made since the bridge was built.

Part Four contains documents and excerpts from important scientific and engineering analyses, reports and studies relating to those issues that might be considered to be controlling in establishing the capabilities of the bridge to withstand forces of nature.

Part Five treats the specific subject of an earthquake that occurred in the San Francisco area on March 22, 1957.

Part Six provides a resume of the wide experience and qualifications of the engineers and scientists who have participated in original bridge design and construction, and in preparing the subsequent technical reports referred to herein.

Part Seven contains a set of maps and drawings relating to the bridge structure and natural phenomena discussed in the reports, covered elsewhere in this report.

Design Characteristics

Part Two provides a review of the important considerations relating to design and construction of the bridge. Analysis of this data should provide ample basis for establishing soundness of structural design.

Aerodynamic Wind Action

There has been only one occasion when action of the elements has caused any real concern. On December 1, 1951, there was a strong southwest wind reaching a peak average velocity of 55 miles per hour, with short gusts of from 60 to 69 miles per hour. As a result of the sustained high-velocity broadside wind, vibrations of unusual amplitude were set up in the suspended center span. Traffic on the bridge was halted for a short period of time during maximum wind intensity, not due to apprehension as to the structure, but due to extreme difficulty of drivers controlling their cars under the heavy wind pressure.

Mr. Clifford E. Paine, noted Bridge Engineer, was summoned from his home in Fennville, Michigan, to make an inspection to determine the effects of the wind, and to supervise any repairs that might be found necessary.

It was found that there had been no damage of any structural consequence, except at expansion joints where the lateral system of the center span connects with the towers. These joints were designed to absorb the stresses of such wind action by use of sliding blocks and bearing plates located beneath the roadway. To improve the mechanical design for future occasions, carbon steel was installed to replace the bronze shoes that had originally been used, with a re-design of moving parts to reduce the amount of dry friction between steel and bronze.

Technical Study by Board of Engineers

In accordance with its usual practice of pursuing all reasonable means of safeguarding the bridge, the Board of Directors on January 18, 1952, concurred in the recommendation of Mr. Paine that a study be initiated to determine the feasibility and desirability of adding a system of bottom lateral bracing to the bridge for added stability against damage that might result from winds of greater velocity, should they ever occur.

Mr. Paine was authorized to proceed and on April 25, 1952, the Board of Directors appointed Mr. Othmar H. Ammann and Mr. Charles E. Andrew, eminent consulting engineers, to collaborate with Mr. Paine as an engineering board to conduct a study.

The problem was recognized as being extremely complex, with its solution dependent upon many varied factors of an uncertain and unexplored nature. Much data on the subject had been compiled since the Bridge was built.

A comprehensive program of section model tests was carried out at the University of Washington. Scale models of the suspended floor structure were tested to determine, by wind tunnel action, the motions that would occur due to dynamic wind action on the structure as originally designed, and as they would exist after stiffening by installation of bottom lateral bracing.

Bottom Lateral Bracing Installed

Upon completion of their studies, the engineering board submitted its report with covering letter to the Board of Directors under date of January 2, 1953. Copy of the letter and the conclusions and recommendations are contained in Part Three as Section A. The report recommended that the District proceed to have a bottom lateral bracing system installed, at an estimated cost of $3,500,000.

The Board authorized the project and appropriated the necessary funds. Competitive bids were called and the work was commenced and carried out under direct supervision of Mr. Paine, and installation was completed in September 1954.

It has resulted in an increase of 35 times in the torsional rigidity of the bridge floor structure, and an increase in over-all torsional resistance of the entire suspended structure including deck and cables, of 2.75 times the corresponding resistance without lower laterals. Photographs of the bridge before and after the stiffening are shown in Part Seven.

Geological and Seismological Study

As a further precautionary move, the Board of Directors authorized the appointment of a board of three scientists from the University of California, renowned in the fields of geology and seismology, to make a survey of conditions as related to bridge foundations and anchorages.

By report dated June 20, 1955, that board submitted its findings, a copy of which is contained in Part Four. In the report, it was stated:

> "We conclude that scrutiny of areas adjoining both north and south foundations of the Bridge and of the Bridge itself over a period of more than twenty years fails to reveal signs of surface instability.
>
> "The observed behavior suggests that both foundation areas are essentially stable and no more likely to slide under the impact of future earthquakes than other areas of comparable relief in the Bay Area."

Again in 1958, the District requested these same scientists to conduct another survey and report their findings. During the interim since their first report, there had been an earthquake of moderate intensity, occurring on March 22, 1957. The Board was interested in knowing whether that tremor had produced any evidence relative to bridge stability.

In their second report of April 4, 1958, copy of which is contained in Part Four as Section D, Professors Turner and Byerly reported, in part:

1. "Again we state that the seismological record offers no indication of an active fault nearer the Bridge than the San Andreas zone."
2. Referring to bridge movement due to the quake, they said:

   > "The double amplitudes there recorded -- 5.6 inches and 2.9 inches -- are trivial displacements compared with those to which the Bridge is repeatedly subjected for much longer times under heavy gales of wind."

3. "Our opinion of June 1955 . . . stands without modification. . . . All the conclusions stated in our report of 20 June 1955 . . . remain valid."

4. "In conformity with prediction, the direct effect of the earthquake in setting up vibrations in the Bridge was trivial; and there is no evidence whatever that the earthquake was accompanied by sliding or by other displacements of basement rock in the vicinity of the Bridge foundations."

5. "We re-endorse and confirm our report of 20 June 1955 in every detail, without revision, modification or qualification of any kind."

Attention is directed to Section D of Part Three, to which is attached a tabulation of bridge vibration data as recorded by modified seismographs located on the bridge. It will be noted that the double amplitude vibration during the earthquake was only 5.6 inches as compared with 18 inches under a moderate wind of from 20 to 25 miles per hour.

Submarine Survey

To supplement the survey of geologic and seismologic conditions, the District retained the services of a consultant in geodesy to make a submarine survey of conditions as related to bridge stability. A report was submitted on April 4, 1958, relating the findings. A copy is contained in Part Four as Section C.

In that report Admiral Gibson, former Hydrographic and Geodetic Engineer for the U. S. Coast and Geodetic Survey, found as follows:

1. "The distance between Fort Point on the south side of the channel near the south tower of the Golden Gate Bridge, and Lime Point near the north tower of the bridge, is essentially the same now as in 1859, indicating no widening of the Golden Gate through sliding, scouring or earthquake faulting."

2. "Neither the submarine troughs, their surface, or bedrock counterparts may reasonably be interpreted as faults. There are no submarine formations that may be construed as confirmation of the submarine faults postulated by Bailey Willis in 1934."

3. "Examination of recorded data on the submarine topography of the Golden Gate channel indicates that there has been no slumping, sliding or faulting. Absence of such evidence tends to confirm the stability of the Golden Gate Bridge towers, piers and foundations."

Reserve Strength of the Bridge

Section G of Part Four contains excerpts from a rail rapid transit study made by a nationally recognized firm of consulting engineers. As one part of the technical studies made by that firm, an analysis of structional characteristics of the Golden Gate Bridge was made.

It was concluded in that analysis that the original design factors of the Golden Gate Bridge were far more generous than would be used if the bridge were to be built today, with the greater experience in design of long span bridges that has been had since 1933, when the design of the Golden Gate Bridge was made. In commenting on the subject, the technical report stated:

1. "Accordingly, the designers had no precedent to help guide them; hence there was a need for conservatism in certain portions of the design. . . . The satisfactory performance of the bridge since it was constructed, together with the conservatism of the original design, form a basis for determining the reserve capacity of the structure."
2. "The discussion above shows a considerable reserve capacity exists in the Golden Gate Bridge."

Design for Earthquakes

Section H of Part Four treats the subject of structural design as related to withstanding earthquake forces. The symposium referred to is rather voluminous and could not reasonably be reproduced herein in its entirety.

Nevertheless, the theme of the entire discussion among the noted engineers and scientists who participated, was that proper structural design, and closely coded and inspected construction, eliminates the prospect of serious damage from earthquake forces of any magnitude with which we are familiar.

Steel structures are far less subject to seismic damage than composite structures, such as found in residential buildings and large office buildings, with heterogeneous composition.

The Golden Gate Bridge is a massive steel structure, homogeneous in character with built-in elasticity as an inherent part of its basic design. It was designed specifically to withstand earthquake shocks. Its foundations are monolithic blocks of concrete resting on solid rock which is far less subject to serious vibration than other earth materials.

A Rational Basis of Earthquake Insurance

Under Part Four in Section F, there has been included an address made by Dr. Bailey Willis, professor at Stanford University, on the subject of earthquakes and insurance. There is much material in that document that is of special interest with respect to the theme of this symposium.

Of particular interest are the following remarks:

1. "Should all buildings be insured? (Referring to earthquake insurance) By no means. There are those which are so located or so constructed that they need no earthquake insurance, as, for instance, a structure deeply founded on solid rock . . . . . "
2. "On the other hand, there are buildings which are so poorly constructed that no insurance company should take the risk"
3. "It is evident that rational insurance demands discrimination among earthquake hazards as among fire hazards."
4. "There is a wide range of risk, from safety to probability of destruction, depending on the firmness of the foundation material, whether it be rock, compacted soil or soft mud."

5. "The materials of which a structure is built are usually given considerable weight in rating schedule, -- but they are not as important as good design and good construction."

6. "Steel is recognized as the aristocrat of structural materials."

Certainly these remarks by such a learned man, made in 1926, ten years before the Golden Gate Bridge was built, would serve two important purposes. First, the basic theories of earthquake design were well known when the bridge was built, and secondly, the features of design are wholly compatible with the theory as expressed so effectively by Dr. Willis.

Summary of Conclusions

Insurance rates must be related to degree of risk. What are those elements of risk? They theoretically involve only five possibilities -- damage by earthquake; damage by geological transformation; damage by dynamic wind action; possible structural failure due to inadequate factors of safety or unknown quantities in design; and the possible effects of enemy attack.

The first three are related to damage by the elements of nature. They have been conclusively ruled out as primary probabilities by original factors of safety in design and subsequent bottom lateral bracing, as well as years of successful experience, as attested by opinion of recognized scientific authorities.

As to possible weaknesses in design, there is nothing to fear since ample factors of safety were employed as demonstrated in the Chief Engineer's Report of September 1937 and the Engineering Board's Report of January 2, 1953.

The fifth risk , that of war, is indeterminate. It is reasonable to predict, however, that a nuclear destructive force of sufficient magnitude to destroy the bridge would probably leave nothing standing in San Francisco or surrounding areas.

It can therefore be concluded, without any question of doubt, based upon the scientific findings contained herein and supported by others too voluminous to include, that:

1. The Golden Gate Bridge is structurally sound, specifically designed to sway with the punches of nature and to harmlessly absorb the movements of the earth crust under natural forces of seismic disturbances.

2. The geologic conditions of the rock structure supporting the two great towers and the massive anchorages, is sound, durable and permanent.

3. Risk of damage to this bridge through any possible source, is no greater than as applied to the many other long span bridges that have been built subsequently, and the type of structure itself inherently places the bridge in a more favorable status than the multitude of office buildings and large multiple dwelling structures of the San Francisco Bay area, when contemplating possible damage by forces of the elements, natural phenomena or hostile enemy action.

James Adam
General Manager
Golden Gate Bridge and Highway District

PART TWO

DESIGN AND STRUCTURAL CHARACTERISTICS

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PART TWO

DESIGN AND STRUCTURAL CHARACTERISTICS

A - HISTORIC RESUME

Initial Survey of Bridge Feasibility

For many years a bridge across the Golden Gate had been considered, but it was not until 1918 that a resolution was introduced in San Francisco authorizing a survey by the city to determine the physical conditions at the site. During that same year the city contacted Mr. Joseph B. Strauss to inquire if he would be interested in the problem. At that time, except for the low level railroad bridge across the Dumbarton narrows on the south end of San Francisco Bay, there were no bridges in the area. All transbay traffic was carried by ferry boats.

At the Golden Gate there were many and difficult problems confronting bridge builders. The channel was more than 300 feet deep and 5,357 feet wide at its narrowest point. Conditions indicated a main span of 4,000 feet, far exceeding anything thus far contemplated in bridge design. The locality was subject to fog and high prevailing winds. It was exposed to the sweep of ocean storms and heavy swells from the Pacific, with a tidal current of seven knots.

The site lay within two military reservations. On the south was the San Francisco Presidio and on the north, Fort Baker. There was strong military objection to such a bridge. The minimum vertical clearance would be greater than had ever been required for previous bridges over navigable waters.

These considerations, and the unprecedented size of the various component units of the structure, the vast quantities of materials to be assembled and transported to their rather inaccessible places of use, the new and original methods of construction in virtually an open sea, combined with the practical limits of time and cost, all presented a formidable challenge to the engineer.

After preliminary surveys, Mr. Strauss submitted his first sketch and estimate on June 28, 1921, embracing a symmetrical cantilever suspension design, with 1,320 foot anchor arms and a 4,010 foot channel span. Clearance of 200 feet was provided and towers were to be 800 feet high. Deck width of 80 feet was provided to allow space for four vehicular traffic lanes, two tracks for a railway and two 7 foot sidewalks. Total cost, exclusive of interest during construction, was estimated at $27,000,000.

Bridge and Highway District Act

On May 25, 1923, the Bridge and Highway District Act became law, and the District was set up including San Francisco, Marin, Sonoma and Del Norte Counties, and parts of Napa and Mendocino Counties. On July 24, 1929, a tax rate was set to obtain funds for preliminary studies, yielding about $465,000.

Appointment of Chief Engineer and Staff

Under contract of October 7, 1929, Mr. Strauss was appointed as Chief Engineer in charge of engineering work, including architectural design, geology, traffic, inspection and supervision of construction. Mr. O. H. Ammann and Mr. Leon S. Moisseiff of New York were appointed as consulting engineers, and later Mr. Charles Derleth, Jr., of the University of California was also appointed as a consulting engineer. These four men acted as an Advisory Engineering Board.

As Geologist, Professor Andrew C. Lawson of the University of California was appointed. Later Allan E. Sedgwick was added as consulting geologist.

In 1931, Mr. Clifford E. Paine was appointed as Principal Assistant Engineer on the Golden Gate Bridge. On February 15, 1933, Mr. Russell G. Cone, whom the Chief Engineer had earlier appointed as Resident Engineer, entered upon his duties in the field at San Francisco, with Mr. Theodore M. Kuss as his assistant. At the same time Mr. W. J. Evans was placed in charge of the surveying staff in the field. These men with their staff were housed in the field offices at the site.

Inspection, being a highly important function of the Chief Engineer, was organized with great care. After thorough consideration Mr. Herbert J. Baker was selected to organize and head the District's eastern inspection staff, with headquarters in New York City. This staff inspected the work performed in the eastern mills and shops. Inspection in the western mills and shops wasplaced with Smith, Emery and Company of San Francisco.

The total engineering force, including designers, detailers, surveyors and inspectors numbered approximately one hundred men. This staff, operating under the direction of the Chief Engineer, handled all of the work on the project from its inception to its conclusion, with one exception, namely that on February 1, 1933, the Board of Directors by resolution transferred the planning of the Sausalito Lateral work to the California State Highway Commission.

Adoption of Specifications

The first meeting of the Board of Engineers was held in San Francisco during August 1929. This meeting resulted in the adoption of design specifications and definite methods of procedure, including among other things the decision to carry out immediately the necessary exploratory drilling and soil tests in connection with the foundations and anchorages. In the interval which had elapsed, any advantages possessed by the cantilever-suspension type bridge had practically disappeared and on recommendation of the Chief Engineer, the cantilever-suspension type was abandoned in favor of the simple suspension type.

The following month preliminary surveys were started in connection with the bridge approaches in the Presidio and Fort Baker military reservations. Negotiations were concluded with the Highway Commission whereby they would undertake the construction of the Waldo Point and Funston Avenue roads as a part of the State Highway system.

Bids were invited for borings at the site of the south pier a short time later and the E. J. Longyear Exploration Company was awarded the contract. Meanwhile, Geologist Andrew C. Lawson had begun his geological studies, and preparations were made for carrying out the soil pressure test behind Old Fort Point, the results of which the reader may have noted in his reading of the section on Geology.

The second session of the Engineering Board was held in San Francisco in February, 1930. Results of the borings, soil pressure test and other foundation data gathered since the Board's previous session were examined, and it was concluded that foundation conditions throughout were entirely satisfactory. It was at this session that the Board approved the Chief Engineer's recommendation that the span be increased to 4,200 feet in order to permit the location of the north pier on a projecting ledge at the Marin shoreline. This would minimize underwater work.

The Chief Engineer presented his outline studies of the general design, the architectural treatment and the structural approaches -- the latter based on non-interference with Old Fort Point. The main dimensions of the structure were fixed in accordance with this design, remaining substantially the same as those of the original design, except that the distance center to center of main trusses, was increased from 80 feet to 90 feet. The original type of floor system was retained, but the surface tracks provided in the original design were eliminated. It was felt that the era of surface cars as a means of rapid mass transportation had passed. This provided six clear 10-foot traffic lanes and two 10-foot sidewalks. Tentative design specifications were agreed upon.

War Department Permit

The hearing for the final War Department permit was set for June 30, 1930, in the Board of Supervisors' Chambers, San Francisco. The hearing was conducted by a Special Army Board comprising three high-ranking officers of the Engineering Corps appointed by the Secretary of War. The District's case, conducted by the Chief Engineer, was supported by a battery of experts, each a distinguished specialist in his own particular line. The protestants did not contest the horizontal clearance. They merely questioned the adequacy of the vertical clearance. On August 11, 1930, the War Department issued its permit on the basis of a 4,200-foot span and a vertical clearance of 220 feet at mid-span and 210 feet at the towers.

Chief Engineer's Report

These preliminaries paved the way for preparation of the Chief Engineer's official report to the Directors. The report briefly described the project, summarized the data gathered since the writer's appointment as Chief Engineer and included general plans, architectural sketches, layout of approach roads, terminals and Toll Plaza, estimates of cost and forecasts of traffic and revenue. In short, this report covered the entire project. It was completed August 27, 1930, and issued in three volumes -- Volume I being the printed report proper, with Synopsis; Volume II, the printed report on traffic matters, and Volume III, the drawings.

The report showed an estimated construction cost of $27,165,000. The Board of Directors of the District concluded that the proceeds of a $35,000,000 bond issue would be ample to meet this construction cost plus a liberal allowance for administration, engineering and financial costs. It was accordingly decided that the project should be presented to the voters of the District in the form and manner as described in the Chief Engineer's official report of August 27, 1930.

Opposing Factions

While these steps were being taken by the District, the opposition had organized a body known as the Citizens' Committee against the Golden Gate Bridge bonds.

Directors and officers were subjected to a constant campaign designed to shake their confidence and break down their resistance. The committee's propagandists claimed that a bridge across the Golden Gate would mar the natural beauty of San Francisco s world famed harbor entrance, that it would destroy Sausalito's splendid isolation, and that that city would be over-run by week end picnickers. The earthquake hazard was enlarged upon and the false information which had been circulated regarding the south pier site was expanded.

The picture painted of Marin was that of a sparsely settled territory which would not support sufficient traffic to permit a toll bridge to meet its financial obligations. Constantly reiterated throughout the whole campaign was the charge that the cost of the project was grossly underestimated. This thought was so deeply bedded in the minds of the public that it persisted almost up until the time the bridge was opened to traffic. Articles in the local and eastern press spread the story throughout the nation until the public generally came to believe that the bridge could not and would not be built.

Bond Election Favorable

Despite this organized opposition, the electors of the District, on November 4, 1930, by a majority well over the two-thirds required by law, voted approval of the issuance of $35,000,000 face value of Golden Gate Bridge and Highway District bonds.

With the District's bonds approved by the unprecedented majority of 145,057 to 46,954, the way now seemed cleared to start the project without further delay, and the Chief Engineer was so directed. Accordingly, arrangements were made with the Coast and Geodetic Survey for a triangulation survey, and conferences were held with the State Highway Engineer in connection with the State's program for building the Waldo Point and Funston Avenue approach roads, and with the military authorities in determining the rights of way for these roads within the Fort Baker and Presidio reservations.

Original Bids

The Directors ordered the Chief Engineer to prepare plans and specifications covering all units of the work so that lump sum bids could be received and the construction cost of the project ascertained in advance of the awarding of any contracts.

Bids were advertised for as returnable in July, 1931. The following tabulation summarizes by unit numbers and character of work covered, the low bids received on that date:

I-A	Steel Superstructure	$10,494,000.00
I-B	Steel Cables, Suspenders & Accessories	6,255,767.65
II	San Francisco Pier and Fender & Marin Pier	2,260,000.00
III	Anchorages & Piers of Approach Spans	1,645,841.28
IV	Steel Superstructure, San Francisco and Marin Approaches	996,000.00
V	Presidio Approach Road	996,180.00
VI	Sausalito Approach Road	67,586.00
VII	Paving of Main Span, Side & Approach Spans	345,000.00
VIII	Electrical Work	133,495.00
IX	Toll Houses & Service Buildings	71,430.00
X	Cement (Estimated 500,000 bbls.)	1,220,000.00
	Total	$24,455,299.93

In addition to the above, there were certain minor miscellaneous items which could of necessity only be covered by estimates. The Chief Engineer's estimate of construction cost, as contained in his Report of 1930 totalled $27,165,000. In the light of the bona-fide bids above noted, it was indisputably evident that the total cost would not exceed the Chief Engineer's estimate.

Final Bids

Original bids were well within the estimates, but contracts could not be let until further legal processes were concluded. It was necessary that the California Supreme Court first clarify certain constitutional questions relative to the taxing powers of the District. This action was favorable.

New litigation developed in a taxpayers' suit in the U. S. District Court. In the spring of 1932, the Bridge District won the suit and litigation was terminated in July 1932. After a further short delay due to financing problems, new bids were received October 14, 1932, and in November 1932, contracts were awarded as follows:

I-A	Steel Superstructure (McClintic-Marshall Corporation)	$10,494,000
I-B	Steel Cables, Suspenders & Accessories (John A. Roebling's Sons Company)	5,855,000
II	San Francisco Pier and Fender & Marin Pier (Pacific Bridge Company)	2,935,000
III	Anchorages & Piers of Approach Spans (Barrett & Hilp)	1,859,855
IV	Steel Superstructure, San Francisco and Marin Approaches (J. H. Pomeroy & Co., Inc., and Raymond Concrete Pile Company)	934,800
V	Presidio Approach Road (Eaton & Smith)	996,000
VI	Sausalito Approach Road	59,780
VII	Paving of Main Spans (Barrett & Hilp and Pacific Bridge Co.)	555,000
VIII	Electrical Work (Alta Electric & Mechanical Company, Inc.)	154,470
	Total	$23,843,905

In November, 1932, contracts were accordingly awarded on the basis of these bids for I-B, II, III, IV and VIII and tentatively for V. Contract VI (the Sausalito Lateral) was not awarded and was later built as a W. P. A. project, as already stated.

Construction

Construction officially was commenced on January 5, 1933, with ground breaking ceremonies at Crissy Field on the Presidio on February 26, 1933.

An agreement was entered into with the California State Highway Commission for planning of the Sausalito Lateral road, in order to facilitate its tie in with the Waldo Point road, and its construction in accordance with State highway design standards. The Sausalito Lateral was constructed as a W. P. A. project. Waldo Point road and Funston Avenue approach road were financed and built by the State.

No special problems had to be met in building of the Marin pier. It was necessary only to enclose the area on three sides by a cofferdam of steel sheet piling buttressed within by rock-filled timber cribbing, maintain the pumps in continuous operation and proceed.

But the building of the San Francisco pier was quite a different story, due to the exposed storm-swept location of the bridge site -- the heavy ground swells, prevailing high-velocity cross winds and tidal currents running upwards of seven knots. The San Francisco pier, 1,125 feet offshore, lies virtually in the open sea and is wholly unprotected from the elements. The contractor's first move toward the construction of this unit was to build an access trestle 22 feet wide and 1,100 feet long from the San Francisco shore to the pier site, bombing each bent shoe into the solid rock of the ocean's floor as construction progressed.

The access trestle had hardly been completed when a vessel off its course in a thick fog crashed through it, not without considerable damage to itself, and the resulting breach had no sooner been repaired when a section of the structure 800 feet long was carried away in a storm. The trestle was then raised five feet and securely guyed at intervals to the bedrock by anchored, steel cables. Thus reinforced, it stood without further mishap until the end of the job, serving effectively over the four year construction period as a means of access for the contractors not only for the San Francisco pier but for the steel superstructure as well.

The fender was an original conception designed to play the dual role of a cofferdam within which the excavation and building of the pier could be carried on and a permanent fender to protect the pier from the impact of passing vessels. The pier proper was to be built within this fender by means of a pneumatic caisson. The contractor had planned to float the caisson into position through an opening left in the east end of the fender for that purpose, the opening to be closed as soon as possible after the caisson was inside. The caisson was built at the Oakland yard of the Moore Dry Dock Company and successfully floated into place, but by nightfall of that same day, under the influence of unexpected heavy swells of unknown origin, it became a battering ram which threatened to demolish the fender wall. At a midnight conference the Chief Engineer gave the contractor permission to remove the caisson, a hazardous job, which was, however, successfully accomplished.

The fender ring was then closed, the inside filled with tremie concrete up to elevation — 35; using the fender as a cofferdam, the water was pumped out and the building of the upper portion of the pier was completed in the dry. This was not done without hazard, as was evidenced in December, 1934, when the prow of a great ocean freighter suddenly loomed out of the fog with but a few feet to spare. Nevertheless, after 20 months' patient labor, including 11 months' delay, the pier was completed.

Subsequent to the bond election, the opponents' criticism of the south pier foundation persisted and in order to quiet this the Board of Directors authorized a further and separate investigation of the foundation conditions existing at the pier site, and upon completion of this investigation accepted the conclusions of its consulting geologists Andrew C. Lawson and Allan E. Sedgwick, which confirmed that the safety of the foundation was beyond question. Nevertheless, after the pier was well along towards completion, a new attack was made by a local and meddlesome geologist, and this was given such widespread publicity that the whole subject was again given a further hearing by the Board of Directors, its Engineering Board and consulting geologists, with the result that this further criticism was found to be equally fallacious. This finally ended the insistent campaign to impeach the safety of the San Francisco pier.

During the progress of the work on the south pier, the north pier and tower had been completed, and the work on the two anchorages and pylon groups had proceeded uninterruptedly.

San Francisco Approach

Some time after work on the Presidio Approach road had begun, objection was raised to its terminus at Marina Boulevard, on the ground that the bridge traffic would increase the congestion on the boulevard intolerably. The District was therefore asked to abandon this terminus and divert all the traffic to Lombard Street. After protracted negotiation between the City, the War Department, the Park Commission and the District, an agreement was reached on a compromise solution proposed by the Chief Engineer, whereby the Marina Terminus was retained as planned and a secondary outlet, for truck traffic principally, was carried behind the Palace of Fine Arts joining with the Presidio Approach by means of a braided connection, and connecting with a new diagonal street now called Richardson Avenue, and thence with Lombard Street. This outlet was subsequently built as a W.P.A. project sponsored by the city.

Military Land Acquisition

The building of the two Presidio viaducts and the Marin approaches involved the demolition and reconstruction of more or less elaborate systems of sometimes outmoded military structures and works, under the supervision of the army engineers. The work performed on this phase of the project includes among other things, the construction of several fire control stations, a $125,000 modern powder magazine, a rifle range, extensive machine and other shops and gas stations, drainage and sewerage systems, living quarters and roads.

The District, by the term of its permit to use and occupy certain reservation property, agreed to replace and make good the damage to military structures and facilities wherever there was any interference. This was an equitable and proper return for the valuable concessions given the District in rights of way, but due to governmental requirements it was a most exacting and trying phase of the work, and for the same reason its cost was an item which constantly grew larger.

The Toll Terminal site, located 170 feet south of the south abutment of the bridge, had been agreed to by the Army as originally determined in the preliminary layout plans, but in making the grant, the War Department had limited the area to 200 feet by 300 feet. From the outset, the District protested the insufficiency of this area and finally prevailed upon the War Department to reconsider the matter. Accordingly, in addition to a liaison officer appointed by the War Department to deal with the District, a local Board of Army Engineers was set up for the specific purpose of reviewing this matter as well as the reconstruction of various batteries, the planning of the powder magazine and other matters relating to the work in the Presidio and Fort Baker. This Board helf formal hearings and after many months of negotiations a satisfactory agreement was reached on all major matters involved, including the expansion of the Toll Terminal area from 200 by 300 feet to 350 by 500 feet. Although the latter dimensions were somewhat less than those urged by the District, it was decided to accept them and a revised permit was issued on this basis.

It was necessary to divert Lincoln Boulevard just south of the Toll Plaza and reconstruct approximately a quarter-mile length thereof, depress it and construct an over-pass across it to connect the Presidio Approach road with the Toll Terminal.

Completion and Cost

The work was completed and the bridge ready for vehicular traffic on May 28, 1937, with appropriate ceremonies.

Cost of the bridge was as follows:

Statement of Construction Cost

Marin Pier	$ 436,000
San Francisco Pier and Fender	2,935,000
Shore-end Pylons	295,000
Cable Anchorages and Housings	2,328,000
Main Towers - Steel	6,970,000
Suspended Structure - Steel	3,250,000
Cables, Suspenders and Accessories	5,910,000
Approach Piers and Structural Steel	1,085,000
Paving Suspended Structure and Approaches	800,000
Presidio Road	1,314,000
Sausalito Lateral Road	330,000
Toll Plaza	450,000
Toll Collection and Recording Equipment	72,000
Roadway Lighting and Electrical	270,000
Tower Elevators	60,000
Military Replacements and Improvements	575,000
Miscellaneous and Equipment	45,000
	$27,125,000
Engineering and Inspection	2,050,000
Administrative and Preliminary Expenses less Taxes Collected	423,000
Financing - Net	4,068,000
Surplus	1,334,000
Authorized Bond Issue	$35,000,000

B - PLANNING AND DESIGN

Principal Dimensions and Quantities

General

Total Length of Bridge Including Approach Structure	8,981 ft.
Length of Suspended Structure	6,450 ft.
Length of Main Span	4,200 ft.
Length of Each Side Span	1,125 ft.
Width of Bridge	90 ft.
Width of Roadway Between Curbs	60 ft.
Height of Towers	746 ft.

General (Cont'd)

Clearance Above Mean Lower Low Water	220 ft.
Weight of Main Span Per Lineal Foot	21,300 lbs.
Live Load Capacity Per Lineal Foot	4,000 lbs.
Total Weight on Marin Pier Foundation	264,000,000 lbs.
Total Weight on San Francisco Pier Foundation	726,000,000 lbs.
Weight of Cable Anchorage at Each End of Bridge	240,000,000 lbs.
Deepest Foundation Below Mean Lower Low Water	110 ft.
Maximum Transverse Deflection, Center Span	27.7 ft.
Maximum Downward Deflection, Center Span	10.8 ft.
Maximum Upward Deflection, Center Span.	5.8 ft.

Towers

Height Above Water	746 ft.
Weight of Two Towers	88,800,000 lbs.
Number of Cells at Base, Per Leg (3'-6" x 3 -6")	103
Number of Cells at Top, Per Leg	21
Base Dimensions (Each Leg).	33 ft. x 54 ft.
Load on Tower from Cables	123,000,000 lbs.
Transverse Deflection	12 1/2 in.
Longitudinal Deflection	(Shoreward - 22 in.; Channelward - 18 in.

Cables

Diameter of Cables over Wrapping	36 3/8 in.
Length of One Cable	7,650 ft.
Number of Wires in Each Cable	27,572
Number of Strands in Each Cable	61
Size of Wire (No. 6) Diameter	0.196 in.
Total Length of Wire Used	80,000 miles
Weight of Cables, Suspenders and Accessories	24,500 tons

Concrete

San Francisco Pier and Fender	130,000 cu. yds.
Marin Pier	23,500 cu. yds.
Anchorages, Pylons and Cable Housings	182,000 cu. yds.
Approaches	28,500 cu. yds.
Paving	25,000 cu. yds.
Total	389,000 cu. yds.

Steel

Main Towers	44,400 tons
Suspended Structure	24,000 tons
Anchorages	4,400 tons
Approaches	10,200 tons
Total	83,000 tons

Planning Surveys

Development of plans for large engineering projects begins with investigations and studies to determine which one of several possible developments would prove most desirable. Based upon these studies, usually extensive in scope, the major features of the general plan are established and the project assumes definite form. In the case of large bridges, the studies normally include consideration of crossings at several different feasible sites from which a choice can be made only after a careful weighing of the merits of each.

The procedure in the Golden Gate Bridge project was much simpler because physical conditions at the entrance to San Francisco Bay are such that a bridge joining the two peninsulas should unquestionably be located at the Golden Gate, extending from Fort Point on the South to Lime Point on the North. Obviously, any other location would involve a much longer structure and would introduce additional construction difficulties with no compensating reduction in those present at the site favored.

From the south shoreline, the floor of the strait slopes gently toward the channel for some 1,300 feet but beyond that point the water depth increases rapidly. The foundation for the South Pier, desired to be not less than 20 feet into the rock, could be prepared under air pressure at a depth of 100 feet below the surface of the water. Meeting the above prerequisites placed the pier 1,100 feet out from shore where the maximum water depth at the channel face of the pier is about 80 feet.

On the north shore, the steep slope of the rock bottom prohibited the placing of piers any appreciable distance off-shore. The length of the main span was thus fixed at 4,200 feet since water depth would make the construction cost of piers between these points prohibitive. The natural location of the South Shore-end Pylon at the Fort Point shoreline fixed the length of the side spans at 1,125 feet. The narrow point of land which served so well to reduce the length of the structure left little leeway as to the location of the bridge in the east-west direction, but fortunately it was possible to keep the south cable-anchorage on land along the west shore of the Point where a setting in serpentine rock could be prepared for it.

On the Marin side, the 1,125 foot sidespan had to skirt the high cliff and terminate in a Pylon just in front of the north cable-anchorage. The topography at this point is well suited for the gravity type anchorage finally adopted since the natural slope of the rock's surface follows the slope which the cables take and it was only necessary to cut a suitable niche in the side of the hill to receive the mass of anchorage concrete.

The bridge terminates within military reservations on both sides of the Gate and from these terminals, roadways had to be constructed through the reservations joining the bridge with existing streets and highways. The location of these roads so as to minimize interference with military facilities was a major task. On the San Francisco side a route was worked out which anticipated a future branch extending southward across the Presidio to connect with Funston Avenue. On the Marin side the extremely rugged terrain added to the difficulty of locating a suitable connecting road. The alignment finally selected passed to the westward of Sausalito joining Route 101 at Waldo Point, a distance of 3 1/2 miles. This carries traffic around Sausalito which is desirable. A lateral, branching off to the eastward, connects with Water Street in Sausalito and serves the needs of local traffic.

The road to Waldo Point became the obligation of the California State Highway Commission as an extension of State Route 101. The proposed branch road through the Presidio connecting with Funston Avenue became the obligation of the City of San Francisco and the State.

The War Department by its permit of August 11, 1930, established the minimum clear height above mean higher high water, of 210 feet at the piers and 220 feet at the center of the main span. This requirement of the permit, of course, determined the height of the bridge. Low-steel at mid-span under normal temperature of 70° F. and dead load only would have to be at least 236 feet above mean lower low water in order to give 220 feet clearance above mean higher high water at 110° F. with a live load on the main span averaging 4,000 pounds per lineal foot. The sixteen feet difference allows for 10 feet increased sag in the main span due to a 40° F. rise in temperature plus live load and 6 feet for the difference between mean higher high water and mean lower low water. (The latter is used as datum throughout the work.) With low steel fixed at elevation 236, the elevation of the roadway and cable at mid-span was readily established at 266.70 and 276 respectively. Choosing a cable sag of 470 feet then fixed the height of towers at 746 feet above datum.

As a matter of proper proportioning it was felt that the width of the bridge having a 4,200 foot span should be not less than 90 feet; therefore, this was established as the width center to center of cables and stiffening trusses. With tower-shafts centered under the cables there would be room for a 60-foot roadway to pass through the towers. It was evident that full advantage should be taken of the possibility of having at once a six-lane road at a relatively small added cost. This would be more than offset by the relief it would offer during periods of extraordinary traffic which would occur even during the early days of service; however, it seemed desirable to occupy with curbs, sidewalk and balustrade the fifteen foot strip remaining outside the roadway on each side of the bridge. This provided sidewalks having a clear width of 10 feet. Balconies around the outsides of the towers serve there as continuations of the sidewalks.

The lateral system is placed in the plane of the top chords of the stiffening trusses, and at each floorbeam, knee-braces areprovided to give lateral support to the bottom chords. Panel lengths are 25 feet and at every second panel point the stiffening trusses are connected with the cables by double suspenders (four parts) of wire rope. The latter are looped over cast steel cable-bands on the cables and their free ends, fitted with forged steel sockets, are attached to the webs of vertical truss members where suitable stiffened seats are provided to receive them. Underneath each sidewalk there is provided a horizontal truss designed to support the curb at six-foot intervals against side thrusts that may be delivered to it by traffic.

Geology

San Francisco Bay was once a wide valley, part of the present-day system comprising the valleys of Santa Clara, Sonoma and Napa. The Golden Gate was then a river gorge through which the greater part of the drainage of California flowed as a fresh water stream to the ocean. Gradual subsidence, as it deepened the gorge, permitted the waters of the ocean to invade the valley till it became the 463 square-mile, saltwater-flooded area.

The rocks traversed by the Golden Gate belong to a series of formations known to geologists as the Franciscan Series and comprise many different kinds of rocks, some sedimentary and some of igneous origin. These formations are widespread and extensively exposed throughout the coastal ranges of California.

After the Franciscan Series had accumulated to a thickness of a mile or more throughout a basin coextensive with the coastal ranges, the region was greatly disturbed by igneous intrusions. As a result, two general types of igneous rock are to be found in many localities. One of these is peridotite, now generally altered to serpentine, and the other, basalt. It happens that the south pier of the Golden Gate Bridge is founded on serpentine and the north pier on basalt. The fact that the narrowest part of the Golden Gate - which is the location selected for the bridge - is bounded on one side by basalt and on the other by serpentine, exemplifies the relatively great resistance of these rocks to the ordinary agencies of erosion.

Because of the intense internal shearing which takes place during its alteration by the process of hydration from peridotite, the tensile strength of serpentine is apt to be relatively low. But it is entirely adequate when confined - as it is. The load stress transmitted by the south pier to the foundation rock is less than 150 pounds on the square inch, and this is relatively a small load. This fact was practically demonstrated by experiment at Fort Point on a representative section of serpentine at sea level during the preliminary investigation. The test section, 20 by 20 inches in area, was loaded to the extent of 92 tons, or 460 pounds per square inch, without yielding. Subsequent pressure tests were conducted in the inspection wells, which were built into the pier and used for access to the rock bottom for this purpose and for purposes of inspection. These tests, together with direct examination, fully confirmed the adequacy of the bedrock for the support of the pier.

It is reasonable to assume that once or twice in a century San Francisco will be shaken by a violent earthquake. The trace of San Andreas fault, upon which a sudden slip occurred in 1906 with disastrous results to San Francisco, lies six miles to the west of the bridge site. But there is no evidence that the Golden Gate itself is in danger of a dislocation such that differential movement between the two bridge-ends might be caused. Even when we contemplate the possible destruction of the Golden Gate Bridge by an earthquake of exceptional violence, it should be borne in mind that any earthquake so violent that it would destroy the bridge would also completely destroy San Francisco. Despite this, the city still continues to build and grow, and growth necessarily involves the erection of large and expensive structures.

Earthquake Considerations

While the determination of stresses due to dead load, live load, wind and temperature presented a well defined problem subject to entirely satisfactory solution, it must be admitted that stresses due to seismic forces could not be evaluated with equal certainty. The towers, homogeneous, flexible shafts of steel, are anchored to massive concrete piers which are founded on rock. Although no one can predict just how a flexible shaft of this character will respond to an earthquake, some conclusions can be drawn as to its stability under these forces. In the judgment of engineers who have investigated destructive effects of earthquakes, these towers may be subjected to earthquake vibrations of very small amplitude (a fraction of an inch) and accelerating forces amounting possibly to 5 per cent of gravity. In the completed structure the transverse deflection of the towers under the design wind load is more than ten times any expected movement of the pier tops, and the stresses from transverse wind will be more than double the stresses due to transverse earthquake forces. Due to the great flexibility of the towers in the longitudinal direction, stresses from longitudinal earthquake forces (5 per cent gravity will not exceed 50 per cent of the longitudinal wind stresses.

The influence of earthquake forces on the design of the towers was a determining factor only in proportioning the anchorage to the piers so as to provide against earthquake while the towers were standing alone on the piers without the stabilizing effect of the cables and the suspended structure.

An earthquake acceleration equal to 10 per cent of the acceleration of gravity, which is greater than can reasonably be anticipated, would increase the maximum bearing pressure on the Marin Pier by not more than four tons per square foot, and that on the San Francisco Pier by not more than three tons per square foot.

Basis of Design

General

Since stresses in the principal parts of the suspension spans depend upon loads applied over great lengths, a proper proportioning of the structure would be obtained only if the basis of design made due allowance for this fact. A single, solid line of vehicles with high percentage of trucks will average in weight about 500 pounds per lineal foot. Such a load in each of the six traffic lanes, making a roadway load of 3,000 pounds per foot of bridge, obviously is a congested loading not apt to occur, but one which may be approached should traffic for any reason be blocked from leaving the bridge. In such event, cars will not stand in contact end to end but will be separated by short open spaces which will, of course, materially reduce the load per foot. The two ten foot sidewalks may receive loads of 100 pounds per square foot over small areas but an average of 50 pounds per square foot is ample when applied throughout the structure. Accordingly, it was recognized that 1,000 pounds per lineal foot would suffice for the sidewalks. The basic live load for the suspended spans was therefore taken at 4,000 pounds per lineal foot of bridge. All loads used for design of the Main Structure are as follows:

Live Load

The basic live load for towers, cables and stiffening trusses is 4,000 pounds per lineal foot of bridge and of the length and position to produce maximum stress. Live load stresses for combination with wind stresses from a wind load of 30 pounds per square foot were computed for the above live loading, except that in the case of the center span stiffening trusses, the live load was taken at 2,000 pounds per lineal foot uniformly distributed over the entire center span plus 2,000 pounds per lineal foot of any length and position in the span to produce maximum stress.

The roadway slab and stringers were designed to carry 24 ton trucks having wheels spaced six feet gauge and axles nine feet center to center. Two thirds of the load was assumed to be on the rear axle. Impact was assumed to equal 50% of the live load. The sidewalk slab and stringers were designed for a live load of 100 pounds per square foot.

Floorbeams were designed for roadway load of six 24-ton trucks abreast with a reduction factor of 75% to allow for the large number of loaded lanes. Impact was assumed to equal 25% of the live load. In addition to the roadway load, a uniform live load on the sidewalks amounting to 100 pounds per square foot was included.

Wind Load

Wind pressures used in design were 30 pounds per square foot on the cables and suspended structure and 50 pounds per square foot on the towers. The exposed area used for computing transverse wind pressure was taken as twice the vertical projection of the structure as seen in side elevation. For the longitudinal wind pressure, the exposed area was taken as 1/2 vertical projection of web member of stiffening trusses and floor beam and knee brace.

Temperature

The range of temperature has been assumed to extend from 30° to 110° F. Normal temperature has been taken at 70° F.

Dead Load

The dead load of the center span if uniformly distributed would be as follows:

	Pounds per Lineal Foot
Cables, suspenders and accessories	6,670
Stiffening trusses	3,330
Floor system, curbs and railings	3,830
Bracing	600
Concrete paving	6,470
Conduits and miscellaneous	400
Total as built	21,300

The dead load in the sidespans as built is 21,500 pounds per lineal foot. For purpose of design an assumed uniform dead load of 21,000 pounds per lineal foot was used for all spans.

Unit Stresses

Permissible unit stresses used in the design are as follows:

Towers

	Lbs. Per Sq. Inch
Direct stress in shafts from dead load, live load and temperature changes	
Carbon steel	14,000
Silicon steel	18,000

Towers (Cont'd)	Lbs. Per Sq. Inch
Combined direct and bending stresses in shafts	
Carbon steel	18,000
Silicon steel 7/8 in. plates	20,000
15/16 in. plates	23,000
Maximum	24,000
Bearing, carbon steel	24,000
Bearing, silicon steel	30,000
Bearing, power driven rivets	24,000
Bearing, carbon steel pins, heat treated	27,000
Shear, carbon steel	10,000
Shear, silicon steel	13,000
Shear, carbon steel pins, heat treated	13,500
Tension, power driven rivets	5,000
Suspended Structure	
Stiffening truss chords, silicon steel	
Tension	32,000
Compression	25,000
Stiffening truss web members, carbon steel	
Tension	21,000
Compression	$21{,}000-90\frac{l}{r}$(a)
Lateral Bracing, silicon steel	
Tension, including participation	32,000
Compression, including participation	25,000
Rivets for stiffening trusses and bracing	
Bearing, power driven	27,000
Shear, power driven	15,000
Floorbeams and Stringers	
Tension, carbon steel	16,000
Tension, silicon steel	22,000
Compression, carbon steel	$16{,}000-150\frac{l}{b}$(b)
Compression, silicon steel	$22{,}000-200\frac{l}{b}$(b)
Shear, carbon steel	10,000
Shear, silicon steel	13,000
Bearing, power driven rivets	24,000
Shear, power driven rivets	12,000
All other Structural Members	
Tension, carbon steel	16,000
Tension, silicon steel	24,000
Tension, heat treated eyebars	27,000
Compression, carbon steel	$16{,}000-70\frac{l}{r}$(a) (14,000 Max.)
Compression, silicon steel	$24{,}000-100\frac{l}{r}$(a) (18,000 Max.)

(a) Where l = length in inches of the unsupported length of the member, and r = the radius of gyration in inches.

(b) Where l = the length in inches of the unsupported flange, and b = the flange width in inches.

Suspended Structure (Cont'd)	Lbs. Per Sq. Inch
Bearing, carbon steel	24,000
Bearing, silicon steel	30,000
Bearing, power driven rivets	24,000
Bearing, carbon steel pins, heat treated	27,000
Shear, carbon steel	10,000
Shear, silicon steel	13,000
Shear, power driven rivets	12,000
Shear, carbon steel pins	13,500
Tension, power driven rivets	5,000
Bending on extreme fibres carbon steel pins, heat treated	27,000
Steel Castings	
Bending on extreme fibres	15,000
Bearing	24,000
Direct compression	20,000
Cable Wire, tension	82,000
Heat Treated Bolts, tension	22,000
Bronze, bearing	3,000

The materials used were required to have yield points and ultimate strengths not less than the following, in pounds per square inch:

	Minimum Yield Point	Minimum Ultimate Strength
Rolled carbon steel	36,000	60,000
Rolled silicon steel	45,000	80,000
Rivet Steel	30,000	52,000
Heat-treated pins and bolts	60,000	95,000
Forged steel rollers	40,000	80,000
Cast steel	35,000	65,000
Heat-treated eyebars	50,000	80,000
Cable wire	160,000	220,000

Main Piers

Preliminary foundation studies completed in 1930 were based upon core borings taken on each shore and at the proposed pier sites. These borings, eighteen in all, showed that the North Pier at Lime Point would be founded on basalt containing inclusions, large and small, of radiolarian chert, an adjoining formation into which it was intruded. The rock is very strong and the cores showed it to be uniform in character except for the occasional inclusions of chert which do not in any way detract from its strength.

Borings at the site of the South Pier showed the rock to be of serpentine identical with that exposed on the shore at Fort Point and having a bearing strength not less than that of the rock upon which the previously described bearing test was made. Subsequent to the 1930 borings fourteen additional core borings were taken at this pier site. In most instances the borings penetrated the rock for 60 or 100 feet and in one case for 160 feet to elevation - 251.6. The borings showed conclusively that the serpentine rock foundation for the South Pier was uniform in character for a minimum depth of 160 feet which corresponds to an elevation of 250 feet below sea level.

The design of the Marin Pier, located on the shoreline as it was, involved no noteworthy engineering problem. A U-shaped cofferdam built out from shore about the pier site would permit the excavation of the rock and preparation of the pier foundation at elevation - 20 and the subsequent construction of the pier, to be carried on as a land operation. The pier base was made 80 feet wide and 160 feet long, the length being determined as the minimum which would accommodate the base dimensions of the tower and the width as the minimum required for stability. The maximum pressure on the base with full live load on all spans is 132,000 tons of which 48,000 tons is due to the weight of the pier itself. The resulting average unit pressure is 10.7 tons per square foot. With full live load on the center span and the far side span, at highest temperature, and with transverse wind, the maximum unit pressure on the base of the pier is 17.1 tons per square foot and the minimum is 4.0 tons per square foot.

Anchorages

Although the Preliminary Plans showed tunnel type anchorages on both sides of the Gate, subsequent study showed that it would be unwise to subject the rock on either side to the character of stress which would result from the use of that type of anchorage. Accordingly, gravity anchorages were developed and shown on the Bidding Plans. These anchorages comprise, at each cable-end, a mass of concrete, weighing approximately 60,000 tons set well down into the rock foundation. They are about 150 feet long and have embedded within them heavy eye-bar chains which terminate in a series of upright girders at the rear ends of the anchorages. At the forward end of each anchorage the eye-bar chains protrude from the concrete as 61 pairs of bars arranged to receive the 61 strands which make up the cable. At each cable-end the total cable pull of 62,000,000 pounds is delivered to the anchorage through these 61 strands. On the San Francisco side, at the anchorage, the cable curve has an inclination of 16° 31' with the horizontal.

On the Marin side the corresponding inclination is 17° 40'. These angles therefore give the direction of application of the cable-pull. The 60,000 ton weight of the anchorage mass is sufficient to turn this pull into a resultant thrust downward against the rock foundation with an inclination from the vertical of about 30°. The unit bearing pressure over any portion of any anchorage foundation does not exceed 10 tons per square foot. The disposition of the anchorage mass is such that it would be stable even though the cable pull were increased to twice the design load.

The anchorages are enclosed in reinforced concrete housings, one for the San Francisco side and one for the Marin side. The roof of the housing on the Marin side serves also as the floor of the bridge.

The eye-bar chains which transmit the loads from the cables to the anchorages are arranged in three tiers of 122 bars each. Those of the first tier are pin-connected to the nine upright girders at the rear of the anchorage. Each girder except the center one receives 14 eye-bars, in three groups, 4 at each end and 6 at the center. The eye-bars are heat treated. Those of the first tier are 10 in. by 2 in. and 33 ft. 4 in. long. Those of the second tier are 10 in. by 2 in. and 66 ft. 7 in. long. Those of the third tier (except for two pairs) are 10 in. by 2 1/8 in. and 33 ft. 4 in. long.

Since the aim was to deliver the cable-pull to the girders at the rear end of the anchorage, precautions were taken to minimize the tendency of the embedded eye-bars to unload through bond with concrete as well as by direct bearing of eye-bar heads against the concrete at connections of adjoining bars. Edges of such eye-bar heads were wrapped with mastic 1 1/2 in. in thickness. While it was felt that painting the eye-bars with some covering which would prevent bond with concrete was highly desirable, a suitable material, known to be free of possible injurious effect over a long period of time, was not available. The bars were given two coats of heavy red lead paint.

Main Towers

The steel towers supporting the cables, obviously notable because of their size, also command attention because of the unusually close correlation of architectural and structural requirements that characterized their design. Developed as rigid frames made up of two shafts and cross bracing without diagonals in the panels that comprise the upper 500 ft. of their 700 ft. height, the towers each contain 22,200 tons of steel, about 10 per cent more than was used in the George Washington Bridge towers at New York. The main section at the base of each shaft or leg of the towers has an area of 12,391 sq. in. as compared with 4,500 sq. in. in the Philadelphia-Camden bridge towers of similar make-up and of 5,000 sq. in. in the eight columns that comprise one leg of the George Washington Bridge towers. The Golden Gate tower shafts are thus by far the largest structural steel members ever assembled.

The towers are 690 ft. 4 in. high from the piers to the base of the saddle castings, and rise 746 ft. above the water to the intersection of the main and the side span cable tangents. The center lines of tower shafts are 90 ft. apart, the distance selected as the desired spacing between centers of cables. This spacing and the necessary width of the tower shafts is just sufficient to provide clearance for the 60 ft. roadway through the towers at deck level.

Supporting a 4,200 ft. center span and two 1,125 ft. side spans through two 36 3/8 in. diameter cables, the towers are designed to resist forces of great magnitude caused by dead and live loads, wind and earthquakes. This fact, coupled with the special indeterminate nature of the structures made the design problem a difficult one. Among the contributions to structural practice that came out of the design is a novel application of the Williot diagram, by which strain equations can be written directly without first knowing the stresses that cause the strains.

The general features of tower design were developed concurrently with the architectural treatment, to insure a satisfactory architectural effect without undue cost or structural sacrifice. The architectural treatment is in general modernistic and involves the exclusion of applied decoration and unnecessary elements. Where ornament was desirable, either to enrich certain surfaces or to diversity surfaces unlikely to be perfectly executed over large areas, a simple vertical fluting was used consisting of intersecting plane facets which could be carried out readily in both concrete and steel. Since an ornamental elaboration of minor details, such as hand rails, lighting units, etc., is likely to be inharmonious in a structure predominantly engineering and industrial in character, these features were handled in the spirit of the large design, using principally structural steel shapes.

Where great heights are involved, the common architectural practice of enlarging the scale of details as their distance from the ground increases, tends to emphasize the detail as such at the expense of the whole mass. The opposite principle was employed on towers of the Golden Gate Bridge; the scale of details on the towers decreases with the height.

The main shafts of the towers are built up of plates and angles arranged in cellular form using sections 3 1/2 ft. square. At the base in the longitudinal direction of the bridge, each shaft is 15 cells wide and in the transverse direction, 9 cells wide. The base slab on which each shaft stands is a 33 x 54 ft. steel plate 5 in. thick, made up of 19 separate pieces. As the shafts progress from the base toward the top, the section is decreased by successive omissions of outside cells, until at the top the total number of cells is only 21, arranged in the form of a rectangle composed of 3 cells in the transverse direction and 7 cells in the longitudinal direction.

Below the bridge floor level, transverse bracing between the two shafts is provided in the form of two panels of diagonals. Above the floor level, however, transverse bracing is confined to four portal struts. All bracing members have four webs which are spaced to permit gusset connections direct to web plates of tower shafts; thus the width of bracing members is approximately the width of three cells, or 10 1/2 ft. The portal struts are cased in steel plate enclosures, detailed to carry out the architectural treatment.

The carbon steel specifications required tensile strength of 60,000 to 70,000 lb. per sq. in. and a minimum yield point of 36,000 lb. per sq. in; for the silicon steel a tensile strength of 80,000 to 95,000 lb. per sq. in. and a minimum yield point of 45,000 lb. per sq. in. were specified.

The allowable unit stresses adopted were as follows:

Direct stress in shafts from dead and live loads and temperature changes:		
Carbon steel		14,000 lb. per sq. in.
Silicon steel		18,000 lb. per sq. in.
Combined direct and bending stresses in shafts:		
Carbon steel		18,000 lb. per sq. in.
Silicon steel	7/8 in. plates -	20,000 lb. per sq. in.
Silicon steel	15/16 in. plates -	23,000 lb. per sq. in.
Silicon steel	maximum -	24,000 lb. per sq. ft.

Owing to the magnitude of the towers and the unusual character of the design, it was thought desirable to have a model built for the purpose of checking its general behavior under the different conditions of load to which it would be subjected. The model was built of stainless steel on a scale of 1 to 56. Proportionate loads were applied to the model and unit stresses were measured by strain gauges. The results obtained from the model checked closely with the analytical results.

Cables

The Bidding Plans and Specifications called for cables each composed of 27,572, No. 6, galvanized bridge wires, grouped in 61 strands of 452 wires each. The strands were to be arranged in the conventional manner which, when all in place and before compacting, would result in a hexagonal cross-section with sides at top and bottom. In the construction of other large bridge-cables, difficulty had been experienced in compacting the cables to a circular cross-section. Generally the horizontal diameter of the completed cable was longer than the vertical diameter and often this difference proved sufficient to give some concern in the fitting of cable bands. The contractor made a thorough analysis of the effect of strand arrangement on the shape of the cable and concluded that an arrangement which placed a vertex of the hexagon at the bottom instead of a side, would require a minimum vertical displacement of wires during compaction and result in a final cross-section conforming more closely to the ideal circular section desired.

This new arrangement of the strands which was adopted, carried with it some variation in the number of wires per strand with a view of getting them in the most advantageous position for compacting. While in the former arrangement, strands were placed in the saddles in horizontal layers, in the new arrangement they were placed in vertical tiers. Fillers placed in the spaces between strands at the saddles were designed to hold the strands accurately in position at these control points.

The circular cross-section into which the cables are compacted obtains up to within 15 feet of the saddles at which points the spiral wrapping stops and retaining collars are placed. Between the retaining collars and the saddles the cross-section of the cables undergoes a transition from a circular shape at the former to the hexagonal shape which it takes throughout the length of the saddles. This length of cable in each instance is protected by means of a removable, weatherproof, cylindrical enclosure supported by the retaining collar at one end and by a shroud casting bolted to the end face of the saddle, at the other end. The end connections of this cylindrical enclosure are provided with packing glands and are so detailed as to allow for all changes in direction of the cable due to changes in temperature and loads.

The cable groove throughout the length of the saddle has a weatherproof steel cover plate, except through the central portion where it is surmounted by a small penthouse made of steel plates and angles. Manholes with weatherproof covers in the north and south faces of this penthouse, provide access for inspection of the cable within the saddle. Mounted on pedestals above the house of the two easterly saddles are the rotating airway-beacons requied by the Department of Commerce.

At the shore ends of the sidespans, the cables are "tied down" to the Pylons by means of a group of short suspenders -- six at each point. These, looping over an articulated cable-band, are adjusted to apply to the cable a load of 1,300,000 pounds under normal temperature and with no live load on the bridge. This load will increase to a maximum of 2,360,000 pounds under highest temperature and with full live load on center and far side spans only. By this means, the cable at that point is maintained at the proper elevation. Means are also provided at these "tie-down" points whereby the cables are there held against lateral movement.

Suspended Structure

The roadway and sidewalks of the bridge are directly carried by rolled-beam stringers which are seated on the top flanges of floorbeams. The latter occur at every panel point of the stiffening trusses which spaces them 25 feet apart. At every second panel point, the stiffening trusses are connected with the cables by double suspenders (4 parts) of wire rope 2 11/16 in. in diameter. The floor system and lateral bracing is in the plane of the top chords of the stiffening trusses. The stiffening trusses are 25 feet deep and their bottom chords, as explained elsewhere, are supported laterally by means of kneebraces at each floorbeam.

The floorbeams are 8 ft. 6 1/2 in. deep, 87 ft. 2 1/2 in. long and weigh 23 tons each. Webs are of carbon steel 1/2 in. thick and are spliced at two points. Flanges are of silicon steel and each consists of two angles 8 x 8 x 5/8 in. and two cover plates. The top-flange cover plates are both 20 x 1/2 in. The bottom-flange cover plates are one 20 x 1/2 in. and one 20 x 7/16 in. The floorbeams were cambered 3/4 in. and the ends were milled so that the end connection angles lie in vertical planes under full dead load.

The stiffening truss chords are of silicon steel. The diagonals and verticals are of carbon steel. Top chord sections have a maximum gross cross-sectional area of 145.51 sq. in. made up of two web plates 36 x 1 1/8 in., four angles 8 x 8 x 3/4 in. turned in, and one top cover plate 30 x 5/8 in. Bottom chords have a maximum gross cross-sectional area of 132 sq. in. made up of two web plates 36 x 15/16 in., four angles 8 x 8 x 3/4 in. turned in, and one top cover plate 30 x 5/8. In the side span stiffening trusses, the heaviest top chord section has a gross cross-sectional area of 91.23 sq. in. made up of two web plates 24 x 13/16 in., two top angles 6 x 4 x 5/8 in. turned in, two bottom angles 8 x 6 x 3/4 in. turned in, and one top cover plate 30 x 11/16 in. The heaviest bottom chord section has a gross cross-sectional area of 88.23 sq. in. made up the same as the heaviest top chord section except that the web plates are reduced to 3/4 in. thickness.

Riveted connections of stiffening truss members were designed to develop the full strength of the members connected, at the allowable unit stresses used for their design. Diaphragms were used in the chord sections at every panel point and, in the case of the center span chords, at mid-panel points. Diagonals are box-sections formed of two ship-channels 18 in. x 51.9 lb. except for a few in panels near the end supports which are slightly heavier.

Verticals are built up I-sections. Those at suspender connections are nominal members and consist of one web plate 32 x 3/8 in. with 3/8 in. reinforcing plate, and four angles 7 x 3 1/2 x 1/2 in. Those at points intermediate the suspenders have to carry floorbeam loads and consist of one web plate 32 x 3/4 in. and four angles 7 x 3 1/2 x 9/16. in.

The lateral bracing system is made up of box-members consisting of four or eight angles laced on four sides, designed to act either as tension or compression members. Since chord increments of wind stress are of considerable magnitude care was taken to apply these forces as near as possible to the neutral axes of the chords so as to minimize local bending.

Calculation of stresses in stiffening trusses was by the "deflection" method and the effect of distortion of long suspenders was included. The stifening trusses are exceedingly flexible and it will be of interest to note that unit stresses in the chords due to live load and temperature depend principally upon the chosen depth of trusses since great changes in the cross-sectional area of chord members would have very little effect on the cable deformation. The wind truss of the central span is likewise very flexible. Being more flexible than the loaded cables, there is a considerable transfer of wind load from the floor to the cables through the suspenders. At any suspender point the ratio of the transferred load to the suspender pull is equal to the ratio of the horizontal projection of the suspender to its length. Solution of simultaneous equations based on this relationship, gave the transferred load at each point. The calculated lateral deflection of the center span under a wind pressure of 30 pounds per sq. ft. is 27.7 feet and this is not much affected by changes in the cross-sectional area of the wind chords. The maximum downward deflection of the center span is 10.8 feet and its maximum upward deflection is 5.8 feet.

The combined unit stress in stiffening truss chords from live load, temperature and wind was therefore determined principally by the depth of the stiffening trusses and their distance apart. The use of silicon steel with an allowable unit stress of 32,000 pounds per sq. in. in tension and 25,000 pounds per sq. in. in compression for the severe combination of loads used in the design, met the requirements. The maximum chord stresses occur near the quarter points and of these the wind stress makes up about three-quarters of the total.

Wind velocities at the Golden Gate do not exceed 60 miles per hour and the bridge is not likely ever to experience lateral forces as high as those assumed. Unquestionably the bridge has ample stiffness in both vertical and horizontal directions. It is believed that the depth of the stiffening trusses could have been reduced considerably without detriment.

The wind truss of the center span is connected with the tower at each end through a hinged and sliding joint located on the longitudinal center line of the bridge. This connection is designed to transfer the lateral forces to the tower and at the same time permit free longitudinal motion as well as angular motion in both horizontal and vertical planes. Stops are provided in this connection which limit the longitudinal movement towerward to 18 inches, and channelward to 21 inches, thus allowing a total range of movement of 39 inches. The calculated movements, exclusive of displacements due to longitudinal wind, is 12.5 inches and 16.5 inches respectively.

The roadway slab, carried on stringers spaced 4 ft. 9 in. apart, is seven inches deep. It is reinforced with fabricated reinforcing trusses having an over-all depth of 4 5/8 in. and spaced six inches apart. Top and bottom chords 11/16 in. diameter are arc-welded to webs 7/16 in. diameter in the form of a Warren truss. The bottom of the truss was spaced 1 in. above the top of the stringer by means of short, steel lugs which were arc-welded to both the stringer and the truss. One inch cover of concrete was provided for the bottom chords which allowed 1 3/8 in. cover for the top chords. Longitudinal rods 3/4 in. diameter were placed in the bottom of the slab, four in each panel between stringers. Longitudinal rods 1/2 in diameter were placed in the top of the slab, one over each stringer and four in between. Concrete was specified to have a strength of 4,000 lbs. per sq. in. in 28 days.

No light-weight concrete was used except for the west sidewalk where encasement of conduits increased the volume of concrete, and in order to keep the weight per foot the same as that on the east side, a concrete weighing approximately 100 lb. per cu. ft. was specified.

PART THREE

SUBSEQUENT STRUCTURAL IMPROVEMENTS

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LETTER OF JOINT ENGINEERING BOARD RECOMMENDING PLANS AND SPECIFICATIONS FOR A SYSTEM OF BOTTOM LATERAL BRACING OF BRIDGE

San Francisco, California
January 2, 1953

To The Honorable Board of Directors
Golden Gate Bridge & Highway District

Gentlemen:

We have the honor to present herewith our report on the investigations which you have requested and authorized us to undertake in order to find ways and means to improve the behavior of the Golden Gate Bridge under dynamic wind action.

Our report covers a brief history of the behavior of the bridge under wind action since its completion, a fuller description of its behavior during the severe storm of December First, 1951, which has led to this investigation, an outline of the various improvements of the bridge structure which we have considered, an account of the section model tests which have been made under our direction at the University of Washington as part of the investigation and a full description with cost estimate of the first step we recommend, namely the immediate addition of a bottom lateral system for the full length of the suspension spans.

Following is a summary of our conclusions and recommendations.

Respectfully submitted,
/signed/
Clifford E. Paine, Chairman

Othmar H. Ammann

Charles E. Andrew

Board of Engineers

CONDENSED SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS OF JOINT ENGINEERING BOARD ON BOTTOM LATERAL BRACING OF BRIDGE

Soundness of Structure

As a result of the exceptional storm of December 1, 1951, the Golden Gate Bridge was subjected to the severest oscillating motions since its completion in 1937. The effects of the storm were described by Mr. Clifford E. Paine in his report of January 18, 1952. We concur in his conclusions that the structure is as sound as ever, but that much can be done to improve its behavior in severe wind storms.

Provision for Dynamic Wind Action

Available knowledge derived from the behavior of similar structures and from extensive research and experimental work which has been carried on in this country and elsewhere since the failure of the Tacoma Narrows Bridge in 1940, supplemented by the studies and model tests made as part of our investigations, lead us to conclude that the steadying of the Golden Gate Bridge to prevent future objectionable motions and to secure an ample margin of safety against dynamic wind action is entirely practicable.

The motion which we are most concerned about is the torsional first asymmetric one in which the main span oscillates with a twisting motion in two segments. We believe that the recommended improvements will eliminate this motion within the range of wind velocities likely to occur.

We expect that the recommended improvements will not entirely eliminate the first asymmetric torsional motions in which the main span oscillates with a twisting motion in one segment, but that this motion will be reduced to a point where it will be neither objectionable nor detrimental.

The nature of the problem to determine the extent of the necessary improvements is an extremely complex one, involving factors, such as the character of wind action on the actual structure and damping resistances within the structure, which at the present time are not fully explored. For these reasons it is not possible to determine at this time the extent of the proposed improvements.

We therefore recommend that certain improvements be undertaken immediately and that other improvements be carried out only in case it should develop that prior steps are not entirely adequate. To carry out all proposed improvements at once might involve unnecessary large expenditures and it is our judgment that successive application of such improvements is not only practicable, but can be carried out at no greater cost than if all steps were undertaken at this time.

Recommend Bottom Lateral Bracing

As a first step, we recommend the addition of a system of bottom laterals in the plane of the bottom chords of the stiffening trusses for the entire length of the suspension spans. Such a system will be very effective in steadying the structure, more particularly against torsional motions. We believe this first step will so materially improve the behavior of the bridge under wind action that no further improvements may become necessary.

Our detailed studies indicate that the installation of such a system of bottom laterals is entirely practicable. It is within the safe carrying capacity of the bridge and can be carried out without interference with the flow of traffic over the bridge.

This installation is estimated to involve a total expenditure of approximately $3,500,000. Its completion will require from nine to twelve months time after award of the construction contract.

Second Step If Found Necessary

As a second step to be undertaken in case the first step should prove not entirely adequate, we recommend installation of a system of stays connecting the cables at the center of the main span with the floor structure.

This step would in our opinion materially enhance resistance against asymmetric torsional motions (the motions which ultimately broke the floor-structure of the Tacoma Narrows Bridge) and otherwise help the steadying of the bridge.

We have made detailed studies of a number of such center stay systems for the Golden Gate Bridge, including adjustable "fixed" stays (as applied in the Bronx-Whitestone Bridge) and "hydraulic" stays (as applied in the New Tacoma Narrows Bridge). Either type would be entirely practicable without detrimental effects on the structure.

Plate IV of this report illustrates the type to which we give preference. We estimate that its installation would involve a total expenditure well within $100,000.

Third Step If Required

A third step which, according to the results of our model tests, would assist in steadying the bridge, consists of a modification of the footwalk structure. The existing solid footwalk slabs would be replaced for part or the whole of their width by open steel gratings. This improvement would be very costly, but would be entirely practicable without serious interference with vehicular traffic over the bridge. We believe that a more elaborate program of model tests should be undertaken to determine the scope of such alterations which would produce the best results under all possible wind conditions. These additional studies may be undertaken if and when it should become evident that this third step is at all necessary.

We have given general consideration to the installation of more extensive stay systems than those proposed at the center of the center span. We believe that such systems are not desirable and would not be more effective than the improvements recommended.

Recommended Action

We recommend that detailed plans and specifications for the bottom lateral system be prepared immediately, that thereafter competitive bids be invited and that construction be expedited as speedily as possible.

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Excerpt From Page 14 of Report

Similar calculations made for the Golden Gate Bridge, as reported hereinafter, show that the torsional rigidity of the floor structure by the addition of the proposed lower lateral system is increased about 35 times, and that the torsional resistance of the entire suspended structure, cables and floor structure, is 2.75 times the corresponding resistance without lower laterals.

As verified by the section model tests hereinafter reported, this stiffening effects a substantial increase of the critical wind velocity at which torsional motions of the structure will be initiated. This increased resistance to torsional motions is accompanied by increased structural damping produced by the addition of the lower lateral system.

These considerations lead to the reasonable conclusion that the addition of the bottom lateral system will so materially improve the resistance of the Golden Gate Bridge to wind action, that no other improvements may become necessary.

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WIND AND EARTHQUAKE EFFECTS ON BRIDGES

With Particular Reference to the Suspension Spans of the Golden Gate Bridge

By Clifford E. Paine
Engineer, Golden Gate Bridge and Highway District

May 23, 1955

General

This is a non-technical presentation of some facts regarding the effects of wind and earthquake on bridges. The risk of structural loss or damage from attacks by these elements depends upon the type of structure, its structural sufficiency and the degree of severity of exposure.

It is important to note that the size of the structure is no measure of the relative risk. In fact, a major structure of unusual dimensional characteristics or form is very apt to be a lesser risk (relatively speaking) than a minor structure of ordinary size and form.

This is true because the designers and builders of the former will, of necessity, give the structure special attention, resulting in less probability of any structural weakness. In use, such major structures, of course, receive more dependable maintenance than do the minor structures.

Wind Effects

It is common practice to provide for wind effect by designing a structure to withstand a wind pressure expressed in terms of "pounds per square foot" of the exposed area. The "wind load," thus determined, is treated as a static load, and the resultant stresses in all the members of the structure are computed for that load.

Experience gained during the past fifteen years has resulted in a better understanding of wind effect. In addition to wind effect as a static load, it has been established that an object placed in a wind current may be subjected to forces which, under proper attendant circumstances, will cause it to flutter or vibrate. The failure of the Tacoma Narrows Bridge focused attention on this phenomenon as applied to bridges.

The floor of a suspension bridge (often referred to as the "suspended structure") is a relatively flexible structure simply suspended from the main cables of the bridge. Wind may cause this suspended structure to vibrate in well defined frequencies. Vibrations may be in a vertical plane, or they may be torsional or a combination of the two.

We are mostly concerned with the torsional or twisting vibrations in which, for example, one half of the span is twisting in one direction, and the other half is twisting in the opposite direction. The velocity of wind from a most favorable direction required to build up noteworthy motion of this character depends in a large measure upon the torsional stiffness of the suspended structure.

In the past, it was accepted practice to build single-deck suspension bridges with a single system of lateral bracing placed immediately below the roadway. It is now recognized that the most effective way to gain the added torsional stiffness, necessary in long spans to get a stable structure, is to employ a second system of lateral bracing separated from the first by as great a distance as the construction will permit.

The Golden Gate Bridge, built with a single system of lateral bracing immediately below the roadway, has now been provided with a second lateral system placed twenty-five feet below the first system. The addition of this second system of bracing has increased the torsional stiffness of the suspended structure of the center span to 35 times its original stiffness.

For fifteen years prior to the addition of this bracing, disturbing motion occurred only once. This was on December 1, 1951, when a 50 to 69-mile wind persisted continuously for several hours, with extraordinary uniformity in velocity and direction and at an advantageous angle.

It should be noted that any considerable change in wind velocity or direction results in a decadence of the motion.

A repetition of this extraordinary combination of circumstances would cause no disturbing motion of the character experienced before the bracing was added.

Earthquake Effects

Fundamentally, the principal parts of the bridge are (1) the piers which support it, and (2) the structures which span the spaces between the piers. The piers, usually of concrete or masonry, are firmly founded in the earth's crust so that any movement of the latter will be imparted to the piers.

The bridge spans may be of various types. They may be classified as relatively rigid structures resting upon the piers or as suspension spans, and relatively flexible structures, carried by cables which pass over the tops of the piers. In some concrete bridges, the spanning structures are integrated into the piers so that the latter becomes a functional part of the span.

During an earthquake, the movements of the earth-stratum on which a bridge pier is founded are imparted to the pier. These movements or tremors have vertical as well as horizontal components. We are concerned mostly with the horizontal components.

The earth-stratum applies to the pier whatever forces are necessary to make it move in unison with the former, assuming perfect rigidity. The magnitude of the forces involved depends upon the mass of the structure and the rate at which it must be accelerated.

In the case of a rigid simple span resting upon, and anchored to, the piers, the latter will impress upon the spanning structure the horizontal forces necessary to make it move in unison with the piers. The mass of the entire rigid structure must be accelerated at the rate of acceleration of the earth tremor. A measure of the force required is the product of the mass and the acceleration.

In the case of a suspension bridge with rigid supporting towers, the latter can be regarded as extensions of the piers and, in this discussion, therefore, we describe the bridge as consisting of suspension spans carried by cables supported by and passing over the tops of piers which, in turn, are founded on a stratum of the earth's crust.

The behavior of such a structure when subjected to earthquake is very much different from that of the structure in the preceding paragraph,or, in fact, that of any rigid superstructure such as a cantilever span.

The reason is obvious when one correctly visualizes the suspended structure as a mass suspended like a pendulum or hammock from the pier tops.

It is manifest that under earthquake forces, the mass thus suspended does not have to be accelerated.

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REINFORCING THE GOLDEN GATE BRIDGE

Excerpts From an Article Written for Publication in Technical Journal

By Clifford E. Paine, Consulting Engineer
Fennville, Michigan

Completion of Bottom Lateral Bracing

The close of 1954 saw completed the installation of a system of bottom lateral bracing on the world famous Golden Gate Bridge. The job had many interesting aspects, not only because it involved the installation of a system of bracing extending for a length of 6,500 feet on the world's longest bridge span, but also because it comprised the most extensive use of High Strength Bolts on any single bridge job.

High Wind of 1951

The story starts with a record-breaking windstorm which struck the Golden Gate on December 1, 1951. This storm was unusual in that wind velocities of over 50 miles per hour continued for three hours and its peak reached 69 miles per hour. The effect of this broadside attack on the Golden Gate Bridge was more severe because throughout the storm the direction of the wind remained the same.

In the fourteen years since its completion in 1937, the bridge had weathered many storms. It had taken storms of short duration with gusts of wind reported as high as 75 miles per hour without undue movement. This one was different. The relentless continuation of high velocity and constant angle of attack eventually set up harmonic movements in the structure, which culminated in a mode of torsional vibration of the 4,200 foot span in two segments. In this motion one side of the roadway would be going downward while the opposite side would be going upward.

At the height of the storm the double amplitude of the movement at the quarter-points of the span was on the order of eleven feet. Traffic over the bridge was stopped for three hours. After the storm passed its peak and as the wind velocity dropped to 50 miles per hour, the motion began to decay and within a matter of minutes reduced to a double amplitude of 20 inches.

Damage to Bridge Only Minor

Damage to the bridge was of a minor nature and was limited to the expansion joints at each end of the main span. The writer supervised the making of the necessary repairs and advised the Board of Directors of the Bridge District that a system of bottom lateral bracing should be installed in the planes of the bottom chords of the stiffening trusses; that this bracing thus placed 25 feet below the existing lateral bracing would make the suspended structure many times stiffer in torsion; and that the behavior of the bridge in similar storms would be greatly improved and probably all objectionable movement would be eliminated.

The Board authorized the work and the bracing was installed at a cost of $3,500,000. The work was fabricated and erected by the Judson Pacific-Murphy Corporation of Emeryville, California. Plans and specifications were by the writer, who also supervised the fabrication and erection.

Use of High Strength Bolts

The plans called for all field connections to be made with 1" High Strength Bolts with hardened washers under heads and nuts. The bolts and washers and their installation were specified to meet the requirements of the "Specification for Assembly of Structural Joints Using High Tensile Steel Bolts" approved by Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, and endorsed by the American Institute of Steel Construction.

Bolts were required to be tightened to a tension of between 40,000 and 45,000 lbs. The use of bolts in this instance seemed to offer several advantages over the use of rivets. It is easier to qualify men to operate impact wrenches than it is to get a large number of good riveters. The operation of inserting bolts, torquing bolts, and spot checking, gives more flexibility in the timing and spreading out of the work than is the case with rivets where more workmen are required around each connection to perform the work.

The aforementioned matters have to do with the practical aspects. However, two other considerations contributed to the Engineer's choice of bolts over rivets. These are: (1) More uniform and consistent results can be expected with bolted connections where all the essentials for a good joint are recognized and properly provided. Some of these essentials will be discussed later. (2) The use of bolts makes possible the setting up of a potential source of energy-dissipation which can be available whenever torsional vibrations reach an amplitude in excess of that required to develop the clamping strength of the bolted connections.

Energy Dissipation with Bolts

It can be shown that a bolted connection employing 1" High Strength Bolts in 1 1/16" holes reamed to matching steel templets in both connected parts can dissipate 250 ft. lbs. of energy per bolt per cycle of alternating stress, such as occurs in the diagonal bracing members of the bottom lateral system under discussion during each cycle of torsional vibration. When the friction between the connected parts is overcome, slipping occurs until the bolt is in bearing in both parts.

The relative movement of the parts from bearing in tension to bearing in compression is 1/8", and for a complete alternating stress cycle is twice that, or 1/4". In a single shear connection, the friction per bolt can be taken conservatively at 12,000 lbs. Hence the work done by friction per cycle is 3,000 in. lbs. or 250 ft. lbs.

The importance of this becomes more apparent when it is evaluated for the entire span. In the main span of the Golden Gate Bridge the end connections of the Diagonals only of the newly added bottom lateral system require 15,000 bolts. For the purpose of this discussion, let us assume that in a rare extremity torsional vibrations have built up to such amplitudes that all of these connections are slipping. The total work done thereby per cycle would be 3,750,000 ft. lbs. If the frequency of the vibrations were 12 cycles per minute, the energy dissipated would be 45,000,000 ft. lbs. per minute.

This is exclusive of the energy dissipated at the bolted connections of transverse and longitudinal struts. These combined are able to contribute an additional damping effect equal to that of the diagonals.

A conservative estimate of the energy which all of the bolted connections are capable of dissipating per cycle of vibration is 7,500,000 ft. lbs., or a total of 90,000,000 ft. lbs. per minute if the frequency is 12 cycles per minute.

This is an important factor affecting the amplitude of movement in any extreme situation where torsional vibrations develop to a degree sufficient to cause slipping at the bolted joints. At such times the amplitude builds up until the energy dissipated per cycle is equal to the energy fed into the system per cycle by the exciting force. Hence this added source of energy-dissipation is highly beneficial. It should be noted that the efficiency of the bolted connection is not diminished by this movement.

Conclusion

It is believed that bolted connections using high strength bolts installed with judicial attention to the essentials described herein, will give results equal to or better than those obtained with field driven rivets.

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SPECIAL REPORT ON GOLDEN GATE BRIDGE OSCILLATIONS

June 29, 1956

Honorable Board of Directors
Golden Gate Bridge and Highway District

Gentlemen:

The writer has just completed a study of Golden Gate Bridge vibration charts and wind charts showing bridge behavior in winds since the installation of Bottom Lateral Bracing. The period covered is January 1, 1955, to March 31, 1956.

The attached tabulation gives a good cross-section of the bridge's response to winds which occurred during that period.

The significant facts revealed by the records may be summarized as follows:

1. The oscillations were always vertical -- no twisting. In other words, there were no torsional vibrations.
2. The greatest over-all height of oscillations recorded was 18 inches -- that is 9 inches upward movement plus 9 inches downward movement. This is technically described as 18 inches Double Amplitude.
3. The commonest form of oscillation is that in which the span vibrates in two segments with the north half moving downward while the south half is moving upward -- technically described as "First Asymmetric Vertical" mode. This was generally combined with another mode of oscillation in which the span vibrated as one segment from tower to tower -- technically described as "First Symmetric Vertical" mode. The frequencies of these two modes are sufficiently different to cause conflict between them so that resonance does not develop.
4. The oscillations were so small that users of the bridge, whether driving or walking, were not aware of them.
5. The main objective of the added Bottom Lateral System was elimination of torsional vibration of disturbing magnitude. It should be noted that no motion of this kind has appeared since the bracing was added.

Respectfully submitted,

/s/ Clifford E. Paine

Clifford E. Paine
Engineer

Clifford E. Paine - June 29, 1956

GOLDEN GATE BRIDGE VIBRATIONS

(From records subsequent to installation of Bottom Laterals)

Date	Double Amplitude Inches	Frequency Cycles Per Minute	Wind Velocity Miles Per Hour		Wind Direction	Mode of Vibration
			Average	Gusts		
1- 1-55	18	8	20-25	34	W	FSV
4- 1-55	16	8	20-30	44	W	FSV & FAV
4-18-55	11 1/2	7 1/2	20-25	40	W	FSV & FAV
4-25-55	11	8 1/4	20-35	40-52	SE	FSV & FAV
4-26-55	14 1/2	8	20-25	38	W	FSV & FAV
12-18-55	7	8	25-40	50-68	SE	FSV
12-21-55	8	7 3/4	32	40-50	S	FSV
12-22-55) 12-23-55)	9	8	30-40	60-63	SSW	FSV
1- 7-56	14 1/2	8	40	44	SE	FSV & FAV
2-17-56	12 1/2	7 1/2	20	30-35	W	FSV
3-31-56	8	8	30	-	SW	FSV & FAV
3-22-57 *	5 6/10	-	13	-	S	FSV & FAV
3-18-58	16	8	50	66	SW	FSV & FAV
4- 2-58	12	8	40	57	SSE	FSV & FAV

* Earthquake

FSV - First Symmetrical Vertical   FAV - First Asymmetrical Vertical

Note: Last three items added since date of report.

REPORT OF GENERAL MANAGER ON BRIDGE IMPROVEMENTS

ACCOMPLISHMENTS IN STIFFENING THE GOLDEN GATE BRIDGE AND IMPROVING ITS AERODYNAMIC STABILITY AND SAFETY

November 1956

Introduction

In July 1955, a report was prepared containing a discussion of the elements related to Multi-Risk Insurance coverage on the Golden Gate Bridge in San Francisco, as particularly concerned with increasing the total insurance coverage and reducing the premium rates.

It was based upon then recent structural improvements in the bridge deck, a current geological and seismological report on supporting earth structure, and completion of eighteen years of successful bridge operation without major mishap. Now another year of successful operation has been added, and other important improvements have been made.

It is the purpose of this present report to supplement the prior repory by supplying additional information related to the improvement projects in a more detailed fashion.

Prior Report of 1955

In the report of July 1955, a brief summary was made of the bridge's history, and the early controversies encountered in obtaining the necessary legislation and financing.

A resume was given of the various engineering and geological problems confronting the bridge from time to time, with brief condensation of several important technical and scientific reports that had been made by eminent engineers and scientists in their respective fields.

The end result of the data contained in the 1955 report was to show conclusively that the Golden Gate Bridge, the greatest suspension bridge in the world, had weathered the storm of adversity, both in the fields of human opposition and forces of the elements.

Nothing that has been done subsequent to the original construction has in any way been a reflection upon the ability or integrity of the designers or builders. In all respects the behavior of the bridge since it was erected has been in complete accord with the predictions of those who brought it into being.

High Wind of 1951

The only discordant note that has sounded during the period of nineteen years since the bridge was opened to traffic in 1937, was its reaction during a heavy storm in December 1951, when a strong southwest wind, reaching a peak velocity of 69 miles per hour broadside from the west, introduced bridge deck vibrations of unusual amplitude in the center suspended span.

Although there was some minor damage done as a result of that high wind, it was confined to those moving parts at the expansion joints that were originally designed for the purpose of absorbing such movements, and there is no reason to conclude seriously that major damage would have been done to the bridge itself had the wind not subsided.

In the interest of exercising all prudent measures of public welfare, the bridge was closed to traffic for a period of about three hours, immediately when the maximum amplitude of vibration was experienced.

Remedial Action

Immediately thereafter, the Board of Directors was quick to realize the desirability of having a complete and expert appraisal of the damage and a qualified estimate of the need for taking necessary steps that would preclude the probability of repetition. A Board of Engineers was therefore appointed under the chairmanship of Mr. Clifford E. Paine, eminent bridge engineer, who was assistant to Mr. Joseph B. Strauss, Chief Engineer of the Golden Gate Bridge, during its original design and construction.

An extensive technical analysis was made by this Board to determine, with the greatest possible degree of precision, what means might be adopted for increasing the factor of safety against high velocity wind pressure.

Wind tunnel tests were made at the University of Washington, with scale models incorporating various devices for dampening oscillations under wind pressure.

Upon completion of those studies, it was the recommendation of the Engineering Board that a system of bottom lateral bracing be installed on the bridge as a means of stiffening the deck structure.

This project was approved by the Board of Directors and entailed a cost of approximately $3 1/2 million dollars. It was designed and carried out under the supervision of Engineer Paine. As a result of that expenditure, the torsional rigidity of the bridge deck structure has been increased by 35 times, and an increase was attained in over-all torsional resistance of the entire suspended structure, of 2.75 times.

Stability of Original Design

Original design of the bridge incorporated into its formula all of those contingencies then known to engineers and designers in bridge construction, with a generous factor of safety in detail and unit stresses.

A catenary suspension bridge is inherently an elastic bridge with great flexibility when considering the tremendous masses of steel involved.

The original design of the bridge probably would have proven itself to have been equal to the maximum forces of nature without the necessity for this additional stiffening process.

The actual damage that was done to the expansion joints connecting the center deck span with the supporting towers was a logical consequence of the type of design incorporated into these moving parts. The 1951 experience indicated that a different type of expansion joint with reversed bearing surfaces would be an improvement over the original. This improvement was made immediately.

Authorization of Funds for Stiffening

In the interest of eliminating any possibility of question as to the stability of the bridge or its ability to withstand wind forces equal to, or greater than, those of 1951, the Board of Directors took the position that not even a calculated risk should be taken in protecting this public investment against further damage.

It, therefore, proceeded to authorize the necessary funds for completion of the project of bottom lateral bracing in the shortest time possible. There were no reservations in this respect.

The recommendations of the Engineering Board were carried out in their entirety. The bridge now stands with a much greater factor of safety than that incorporated in its original design, which in itself, due to the magnitude of the project, was considered conservatively ample.

Financial and Structural Stability Proven

Growth of traffic over the Golden Gate Bridge has been increasing at a rapid and continuous rate since the end of World War II. With the high rate of influx of population into California, and the growth of industrial and commercial activity, there has been great residential expansion.

Counties to the North of San Francisco that are served by the Golden Gate Bridge are feeling this high rate of increase, and there is no reason to expect any diminution in the rate of growth within the foreseeable future.

This heavy volume of traffic has made it possible to reduce the cash automobile toll on the bridge since the war from 50¢ to 25¢ per vehicle by successive steps. Even at this low rate of 25¢ and the exceedingly favorable commutation rate of 17 1/2¢ per trip, the financial stability of the bridge is assured for the future.

Therefore, in summarizing, there appears to be no risk with respect to the Golden Gate Bridge, either as to structural stability or financial integrity, within reasonable bounds of prediction.

The only remotely possible sources of major damage would be through forces of nature that are far greater than anything yet experienced, or the catastrophe of enemy attack in time of war.

The unfortunate collapse of the Tacoma Narrows Bridge in 1940 had the effect of creating a grave question in many minds as to the durability of suspension bridges. Actually, the only similarity between that bridge and the Golden Gate Bridge is that they were both single span suspension types.

Discussion of Bottom Lateral Bracing

There are many interesting aspects to the story of the bottom lateral bracing project. It was not a mere routine operation employing common practices and standard materials. It was unique in many respects. The story has been told very effectively by Engineer Paine. The following description is based upon his report.

Prior to installing the bottom lateral bracing, the deck structure of the bridge consisted of a long rectangular box stretched between the towers a distance of 4,200 feet with a width on top of 90 feet, and a depth of 25 feet on the two sides. This large box-like structure was open on the bottom, the same as an inverted box with its lid removed.

Although diagonal bracing was installed at appropriate intervals between the underside of the roadway deck and the vertical sides of the structural box, it inherently had a much higher degree of flexibility with the fourth side absent, than would have been the case if the side were closed. Essentially, the problem was one of placing the lid on the box and trussing it in place so as to form a rigid four-sided structure.

The plans and specifications, as drawn by Mr. Paine, called for all field connections to be made with 1-inch high strength bolts with hardened washers under heads and nuts.(a) The bolts and washers and their installation were specified to meet the requirements of "Specification for Assembly of Structural Joints Using High Tensile Steel Bolts" as approved by the Research Council on Riveted and Bolted Structural Joints, of the Engineering Foundation, and endorsed by the American Institute of Steel Construction. These bolts were required to be tightened to a tension between 40,000 and 45,000 pounds.

It is the conclusion of Mr. Paine that bolted connections, as used, with high strength bolts under careful supervision and rigid specifications, give results equal to or better than those obtained with field driven rivets.

Subsequent Observations as to Bridge Stability

In a report to the Board of Directors dated June 29, 1956, Engineer Paine reported that he had completed a study of the vibration charts and wind charts to compare the wind behavior in winds since the installation of the bottom lateral bracing, over the period January 1, 1955, to March 31, 1956.

The conclusions derived by Mr. Paine from this analysis were briefly as follows:

1. The oscillations were always vertical with no twisting. Torsional vibrations were absent.

2. The greatest over-all double amplitude oscillation recorded during that period was 18 inches.

3. The frequencies of First Asymmetric Vertical mode and the First Symmetric Vertical mode were shown to be sufficiently different to cause conflict between them, thereby preventing the establishment of resonance.

(a) Technical aspects of high strength bolts discussed in Mr. Paine's report on page 42 hereof.

3. (Continued)

In the First Asymmetric Vertical mode, the suspended span vibrates vertically in two segments with one-half raising while the other half moves downward. In the First Symmetric Vertical mode, the entire span moves uniformly as one segment upward and downward.

4. The oscillations were so small in magnitude that users of the bridge, either driving or walking were not aware of the movement.

5. It was the main objective of the bottom lateral system to eliminate torsional vibration of disturbing magnitude and in none of the tests was there any appearance of this sort of motion since the bracing was added.

Installation of Travelling Scaffolds

As a further indication of the policy of the Board of Directors of the Golden Gate Bridge and Highway District of adopting every reasonable means of protecting the bridge structure and increasing the safety of its maintenance workers, another expenditure of approximately $700,000 was made.

Subsequent to installation of the bottom lateral bracing, the Board directed the design, construction and installation of a system of motorized travelling maintenance scaffolds.

These scaffolds travel on rails throughout the length of the suspension structure, a distance of 4,200 feet in the main span, and 1,125 feet on each of the side spans. These scaffolds were also designed and engineered by Mr. Paine.

The scaffolds incorporate every known safety feature for the protection of workmen, with necessary limit switches and interlocking devices to safeguard the units against damage by improper operation.

There are four sets of these scaffolds on the bridge, two on the center span and one on each of the side spans.

One complete scaffold set consists of one interior member, two exterior side members (one on each side of the roadway), and one exterior member underneath the bottom lateral bracing.(a)

All of these members of the composite unit move simultaneously by the application of power by synchronized electric motors. Each scaffold has its own engine generator set to supply the electric power for propulsion.

With this system of specially designed scaffolding, the maximum degree of protection to the bridge structure is insured. It represents just one more important step taken by the Board of Directors to safeguard, by all possible means within reasonable limits, the permanence and stability of this great bridge.

(a) Technical aspects of scaffolds discussed in Mr. Paine's report on page 53 hereof.

This established attitude of the bridge directors in fixing their long-range policy should be of considerable reassurance to the underwriters of Multi-Risk Insurance coverage on the bridge.

Likewise, it is reasonable to expect that these major protective measures should be proportionately reflected in the premium rates for Multi-Risk Insurance.

Respectfully

/s/

James Adam
General Manager
Golden Gate Bridge and
Highway District

REPORT ON MOTORIZED TRAVELLING MAINTENANCE SCAFFOLDS FOR THE GOLDEN GATE BRIDGE

September 25, 1956

General

A system of new motorized travelling maintenance scaffolds has been installed this year on the Golden Gate Bridge -- the world's greatest suspension span.

These scaffolds travel on rails throughout the length of the suspension structure -- 4200 feet in the main span and 1125 feet on each of the side spans. They were designed and engineered by Mr. Clifford E. Paine, Engineer for the Golden Gate Bridge and Highway District, who was associated with Joseph B. Strauss, Chief Engineer of the Bridge from the time of its inception and during the entire construction period.

Incorporated in their construction is every known safety feature for the protection of workmen, and necessary limit switches and inter-locking to safeguard the units against damage by improper operation.

The scaffolds operate in sets, there being one set in each side span and two sets in the center span. A set is made up of one interior scaffold, two exterior side scaffolds (one on each side of the roadway), and one exterior underneath scaffold.

The State Division of Industrial Safety, State of California, was asked to review and discuss the design drawings of the scaffolds before any contract was awarded for their fabrication and erection. The Division reported that the design was especially well suited to counteract the numerous hazards that would otherwise exist in the operation of maintenance scaffolds.

The four complete sets of scaffolds embrace 750 tons of structural steel, including 440 tons of rails and rail chairs, and in addition, operating machinery on the scaffolds weighs 58 tons.

The cost of the scaffolds was approximately $700,000.

Interior Scaffold

The interior scaffold is designed to run on rails which were installed just above the bottom lateral system. The scaffold is motor driven by electrical power derived from its own engine generator set. This scaffold therefore moves in a clear space bounded by the bridge floorbeams above, the bottom lateral system below and on either side (transverse to the bridge) by vertical members which occur at each panel point (twenty-five feet spacing in the longitudinal direction of the bridge). The clear distance between these vertical members is about sixty feet.

Structurally, the interior scaffold consists of three main elements -- one Carrier unit and two Retractile Wings.

The Carrier, as its name implies, is the main unit. It has a top working deck and below deck it carries the Retractile Wings. It runs directly on the rails. Extending in a direction transverse to the bridge, it has a tubular space of rectangular cross section. At the bottom side of this space, rails are provided on which the Retractile Wings can be rolled in or out in direction transverse to the bridge. The scaffold cannot be moved past a panel point unless the Retractile Wings are fully retracted. When the scaffold is in position between panel points, the Retractile Wings, one on each side, may be run out, thus extending the working deck to the inner sides of the Stiffening Trusses. Movement of the Retractile Wings is effected by means of a rope drive motored by electric power.

Necessary limit switches and interlocking is provided to safeguard the units against damage by improper operation and to limit travel to that intended.

Machinery for driving the Carrier is located on the shoreward side only, and it is connected only to wheels at the shoreward end of the trucks. Wheels at the opposite end of the trucks are idlers. Rail clamps are provided adjacent to the drivers with only two per scaffold.

Interior scaffolds for center span and side spans are alike except that the wheels and driving machinery of the side span scaffolds are set lower to compensate for the differences in distance between top of rail and under side of floorbeams in the two spans.

Exterior Side Scaffold

The exterior side scaffolds are in pairs, one right-hand and one left-hand. They hang in vertical planes just outside the stiffening trusses. At the top they are provided with wheels which roll on the crane rails previously described. At the bottom, the two scaffolds forming a pair are joined by two wire ropes which restrain them from outward movement.

These scaffolds are thirty feet long in the longitudinal direction of the bridge and three feet wide. At each end is a latticed upright hanger. At their tops the hangers terminate in trucks which extend inward in cantilever fashion, carrying at the cantilevered end a wheel which rolls on the crane rail. The truck on the shoreward end is designated a Driver Truck because its wheel is driven by electric motor-driven machinery mounted on the truck. The truck on the other end of the scaffold is designated a Trailer Truck because its wheel is not used for propulsion. The transverse wire ropes mentioned above are attached to the bottom ends of the hangers - one to each hanger. About eight feet up from the bottom end of each hanger there is mounted thereon a rubber-tired wheel which in operation rolls along the webs of the bottom chords of the stiffening trusses. These serve to restrain the scaffolds against inward movement. At the upper end of each hanger there is mounted thereon a trolley which functions in a horizontal plane in rolling engagement with the aforementioned trolley beams. These are arranged to hold the wheels in proper lateral relation to the crane rails. Rail clamps located six feet three inches in from the trucks provide means for mooring the scaffold in any position.

Combined with each rail clamp support is a safety stop which just clears the rail by a fraction of an inch. This safety stop is capable of supporting the scaffold in the event of any wheel, truck or rail failure.

The outboard or outer plane of the scaffold is provided with a system of diagonal bracing and the inner face is open except for hand-railings and one vertical tension member on the centerline.

The top deck of this scaffold, which is about one foot nine inches above the top chord of the stiffening truss, serves as an auxiliary operator's platform and a means of access to lower levels. Below the top deck there are four platforms three feet wide and thirty feet long. They are spaced vertically about eight feet apart (the bottom space is greater) to serve as working platforms for maintenance workers.

The ends and back side of the scaffold are covered with woven wire fabric as a safety measure to prevent men or material from falling off the scaffold.

Electric power for operating the exterior side scaffolds comes from an engine generator set on the exterior underneath scaffold. This power is delivered by a flexible power and control cable extending from terminal box on each end of underneath scaffold to cable reel mounted on the low end of each shoreward hanger.

Exterior Underneath Scaffold

The exterior underneath scaffold comprises a working platform eight feet wide, extending the full width of the bridge and supported by two trusses four feet deep. The scaffold is provided with two trolleys per truss spaced forty-five feet center to center, and in rolling engagement with trolley beams now in place just below the Bottom Lateral Bracing of the bridge.

This scaffold is operated in conjunction with the pair of exterior side scaffolds with which it is associated. It is propelled along the bridge by means of wire ropes working over drums (one at each end). The ends of the wire ropes are anchored to each end of the adjacent exterior side scaffold, and the drums are electric motor driven, power being derived from an engine generator set located on the scaffold. A latching device is provided at each end which affords a means for mooring the scaffold to the exterior side scaffolds.

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REPORT ON SAFETY ASPECTS OF TRAVELLING SCAFFOLD

State of California
Department of Industrial Relations
Division of Industrial Safety
965 Mission Street
San Francisco 3, California

November 9, 1956

Golden Gate Bridge and Highway District
Box 99
Presidio Station
San Francisco, California

Attention: Mr. James Adam, General Manager

Gentlemen:

On November 7, 1956, an engineer from this office had an opportunity to inspect the new power-driven rolling scaffolds to be used for painting and maintenance work on the Golden Gate Bridge. They have been carefully designed by Bridge Engineer, Clifford E. Paine, to support, with ample safety factor, all expected loads, and to provide numerous devices for the elimination of hazards normally connected with bridge painting operations.

Some reasons for the opinion, that this is one of the finest scaffold arrangements of its kind are as follows:

1. A synchronizing arrangement for the driving motors makes it unlikely that misalignment of the exterior, side scaffolds will become a serious problem.
2. Large sections of rope netting for added protection are provided under those parts of the scaffold from which the most difficult work will be handled.
3. The exterior, side scaffolds ride on heavy rails, and in addition are provided with guide rollers to positively prevent derailment. Rail locks are provided to hold these and the interior scaffolds in place, with no possibility of movement until the locks are released.
4. Trolley supports for the lower scaffold are so arranged that structural members will come into contact and thus provide emergency support should one or more of the trolley connections fail.

5. Interlocks, limit switches and travel stops are located at critical points to limit the possibility of human error leading to trouble with the operation of power equipment.

6. The outer sides and both ends of all exterior, side scaffolds are covered with heavy wire fabric completely eliminating the possibility that men or material could fall from these protected sides or ends. All other working platforms are completely surrounded with standard, two-rail railings or equivalent.

It is our understanding that the men who are to operate these scaffolds will have a training period of approximately thirty days, and that signs will be posted on the scaffolds specifying the maximum number of men and other loads that can be accommodated.

The bridge administration and the design engineers should be commended for their foresignt in providing safe and modern equipment of this type, and the Division appreciates your interest in arranging for an inspection by our safety engineer.

Very truly yours,

DIVISION OF INDUSTRIAL SAFETY

A. C. Blackman, Chief

/signed/

V. L. White, Supervising
Construction Engineer

PART FOUR

GEOLOGY AND SEISMOLOGY

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REPORT OF THE GEOLOGIST ON EARTH STRUCTURE AND EARTHQUAKES

May 8, 1930

To the Honorable Board of Directors,
Golden Gate Bridge and Highway District,
San Francisco, California.

Attention of the Chief Engineer

Gentlemen:

Geologic History

The Golden Gate at San Francisco, which it is proposed to span by a suspension bridge, was at one time a river gorge through which flowed the greater part of the drainage of California as a fresh water stream. Beyond the Gate a delta was built out into the deep water out as far as the Farallone Islands, and was spread along the coast by the currents and waves for many miles as a great embankment. When the mouth of the Sacramento River was on the surface of the delta outside the gorge, the coast was much higher relatively to sea level than at present.

What is now the Bay of San Francisco was then a broad valley, like that of Santa Clara on the South, or Napa and Sonoma valleys on the north. This physiographic condition was changed by the inauguration of a subsidence of the coast in the region of the outlet of the river and its growing delta. The subsidence, proceeding slowly, allowed the waters of the ocean gradually to invade the river gorge, and then, equally gradually, to expand over the valley through which the river flowed, till it became the magnificent harbor of San Francisco Bay.

The islands in the bay are the former hills of the valley land. The bay became a trap for the sediments brought out by the streams from the Sierra Nevada and from the east side of the Coast Ranges; and the depression of the old valley has been largely filled with that deposit. But the rate of subsidence has been sufficient to prevent the depression from becoming completely filled, and so converting it again into a valley floor.

The total amount of subsidence may be ascertained approximately from the fact that the deepest part of the Golden Gate is 63 fathoms or 378 feet. When the gorge was functioning as a river channel, it is improbable that its bottom was more than 30 feet below sea level. We may conclude, therefore, that the total subsidence has been about 350 feet.

At the present time the subsidence, if still in progress, is so slow that it is not appreciable, and there has been no measurable change of level of bench marks established on rock at San Francisco many years ago by the U. S. Coast and Geodetic Survey.

Thus from the point of view of geological history, the Golden Gate Bridge is a span over a river gorge.

Rock Formation

The rocks traversed by the Golden Gate belong to a series of formations which are wide spread and extensively exposed throughout the Coast Ranges of California. It is known to geologists as the Franciscan Series, and is so named from the City of San Francisco. The series comprises many different kinds of rock, some sedimentary and some igneous. The sedimentary formations consist chiefly of sandstones and shales, which are for the most part non-marine, and are recognized by geologists as continental in origin, having been deposited on the flood plains of rivers or in interior basins.

The sandstones are very strongly indurated as a rule, and are locally metamorphosed into various crystalline schists, among which are bright blue glaucophane schists containing the rare and very hard mineral Lawsonite. These continental formations alternate with marine formations of two distinct types, one known as radiolarian chert and the other as foraminiferal limestone, both well stratified rocks abounding in minute, marine, fossil organisms.

During the deposition of these varied strata, there were some active volcanoes in the region, for we find occasional sheets of lava between the sedimentary formations.

This entire aggregate of stratified rocks and lavas is over a mile in thickness in the vicinity of San Francisco, and their stratigraphic relations are particularly well displayed in the Marin peninsula, which forms the north side of the Golden Gate.

After the Franciscan series had accumulated to the thickness of over a mile in a basin coextensive with the present Coast Ranges, the region was greatly disturbed by igneous intrusions, and as a result, two general types of igneous rock are found intruded into it at many localities. One of these is peridotite, now very generally altered to serpentine, and the other is basalt usually with a pronounced ellipsoidal structure.

South Pier Location

It happens that the south pier of the bridge is founded on serpentine and the north pier on basalt. The fact that the narrowest part of the Golden Gate, that selected for the site of the bridge, is bounded on the one side by serpentine and on the other by basalt, is significant of the relatively great resistance of these rocks to the ordinary agencies of erosion.

The serpentine at Fort Point, on the south side of Golden Gate, is derived from peridotite by a process of hydration, whereby the originally anhydrous silicate of magnesia takes up about 13 per cent of its weight of water, and so greatly increases its volume. To accommodate this enlargement, the rock shears internally as the hydration proceeds, and when the process is complete it consists of an aggregate of residual unsheared, massive spheroids of strong serpentine embedded in a matrix of sheared and schistose serpentine having a very low tensile strength. The residual spheroids vary in diameter from a few inches up to 10 feet or more.

Strength of Supporting Rock

While the tensile strength of this rock is low, under compression and confined, as it will be, it is entirely adequate to support the terminal pier of the bridge. The load stress transmitted from this pier to the foundation rock is less than two hundred pounds to the square inch, and this is a relatively small load. Sand has no tensile strength at all, but under compression and confinement it makes an excellent foundation for structures transmitting a very much larger load stress than is here proposed.

Similarly the serpentine, notwithstanding its low tensile strength, could sustain a much heavier load than that of the bridge pier. This fact was practically demonstrated by the test conducted by Mr. Strauss, Chief Engineer of the Golden Gate Bridge and Highway District, at Fort Point on a representative section of the serpentine. An area of the serpentine, 20 by 20 inches, at sea level was loaded to the extent of 92 tons, or 460 pounds to the square inch, without yielding.

The bearing strength of the foundation rock of the south pier, 1000 feet north of Fort Point, will be increased by the confinement afforded by the excavation made to receive the pier. I recommend that this excavation, at its periphery, be carried to a depth of not less than 25 feet below the natural rock surface of the sea floor measured from the lowest point of its intersection with the pier.

The cores of the borings made at the site of the south pier show that the rock is identical with that so well exposed on shore at Fort Point, and an examination of the cores indicates that the bearing strength of the rock is not less than that of the rock upon which the test was made.

The south anchorage of the bridge will have to be designed to depend upon dead load rather than upon the tensile strength of the rock, or upon the frictional resistance of the latter to the pull of the anchor.

North Pier Location

The north pier of the bridge at Lime Point is founded as basalt containing inclusions, large and small, of radiolarian chert, an adjoining formation into which it was intruded. There are present in this rock the usual joints which affect all rocks and some shears; but the rock is very strong and is amply adequate to sustain the load of the pier. The cores from the drill holes show that it is uniform in character except for the occasional inclusions of radiolarian chert, which do not in any way detract from its strength. Since the subaqueous slope at the pier site is rather steep, I recommend that the foundation excavation be carried to a depth of not less than 25 feet below low tide, and that the south face of the pier be located not less than 20 feet north of the present low water shore line.

The north anchorage is located partly in sandstone and partly in greenstone, a diabase variety of the basalt. Both rocks are all that could be desired for the purposes of anchorage.

Possible Earthquake Occurrence

Once or twice in a century, it may reasonably be assumed, the region of San Francisco will be shaken by a violent earthquake. Six miles to the west of the proposed bridge is the trace of the San Andreas fault upon which a sudden slip occurred in 1906, with disastrous results to the City of San Francisco. Every design for a large structure on San Francisco Bay should take into account the stresses which may be engendered by a repetition of that movement, or by a fault slip elsewhere in the region.

So far as I am aware, there is no danger of a dislocation in the Golden Gate itself, whereby a differential movement of the two ends of the bridge might be caused. The danger to be guarded against is the excessive swaying of the bridge induced by the vibration and commotion of the earth, due to a slip on some fault within a radius of twenty miles. In such an earth movement there are three components of motion to be considered: (1) that parallel to the length of the bridge, (2) that transverse to the bridge, and (3) the vertical component. The parallel movement will throw a sudden strain upon the anchorages, first on one and then on the other, and the strain will cause the bridge between the terminal piers to sway up and down.

Elasticity of Suspension Span

The chief safeguard against damage from this cause is the elasticity of the steel, but the anchorages should be designed to withstand very much greater stresses than those due to the dead load of the bridge -- stresses which may be applied very suddenly and for which the only remedy is strength of structure. A remote contingency, which nevertheless should be considered in the design, is the possibility, in a prologed earthquake, of a longitudinal or north-south swaying of the bridge magnified by resonance. This might happen if the main vibratory movement of the earth were north and south, and the period of the earth waves happened to be the same as that of the vibration of the bridge.

Owing to the nature of the structure and its great elasticity, this longitudinal swing would also almost instantly be transformed into an up and down sway of the mass suspended between the piers. The strain at the anchorages would be in some measure proportionate to the amplitude of this vertical sway.

In the second case, where the chief component of the earth vibration is transverse to the bridge, the effect would be to cause the structure to swing normal to its length. If the bridge be heterogeneous in structure, that is, if it be not built throughout of the same material, it will suffer more than if it be homogeneous. Thus, if the bridge is carried on masonry towers, the masonry will have one period of vibration and the steel another, and the two structures will hammer each other violently. The destructive effect of this reciprocal hammering was well displayed in the destruction of the city hall of San Francisco in 1906.

It is important, therefore, in the design of the structure to make it homogeneous. The transverse swing of the bridge may be augmented by resonance, if the natural period of the structure happens to be the same as that of the earth waves. Since such a resonant swing would have a much greater amplitude than that of the earth waves it might be checked by lateral guys extended on either side, not only from the piers but also from the suspended structure. Any horizontal transverse swing which may be induced in the bridge by an earthquake will cause extraordinary stresses to develop in the piers as the load of the bridge is thrown from one side to the other, and lateral tilting of the piers will be inevitable. This can be minimized by using heavier steel sections than would otherwise be necessary, and by securely anchoring the piers into the concrete foundations.

The third component of the earth motion, the vertical, will be effective chiefly as a hammer blow upward on the base of the piers. Owing to the great inertia of the mass supported by the piers, this sudden blow will tend to buckle the steel of the piers before the upward movement can be transmitted to the structure as a whole. This tendency may be met by making the lower part of the piers stronger and more rigid than would be necessary merely to support the dead load of the bridge.

Relative Prospect of Damage

In general, the risk of damage to the bridge from earthquake shock is not so great as it is in the case of the skyscrapers of San Francisco. The latter are not homogeneous structures, but are built of steel and masonry, and these willnot swing in unison, but will tend to be mutually destructive. In the fling of the strong steel frames, the veneer of masonry will be thrown off in many cases, causing great damage in the streets. The bridge, on the other hand, if built wholly of steel, will have the advantage of homogeneity.

Even when we contemplate the possible destruction of the bridge by an exceptionally violent earthquake, the fact should be borne in mind that the life of a steel bridge near the salt water is limited, and that its life, if never affected by earthquakes, is probably less than the mean interval between heavy, destructive shocks. Any earthquake, so violent that it would destroy the bridge, would also destroy San Francisco. Yet, though it faces possible destruction, San Francisco does not stop growing, and that growth necessarily involves the erection of large and expensive structures.

I trust that the foregoing memorandum may be of service to you.

Respectfully submitted,

/s/ Andrew C. Lawson,

Consulting Geologist

REPORT OF THE ENGINEERING CONSULTANTS ON FAVORABLE BRIDGE DESIGN

July 28, 1930

To the Honorable Board of Directors
Golden Gate Bridge and Highway District
San Francisco, California.

Gentlemen:

Having been appointed by your Honorable Board as members of the Board of Engineers to collaborate with the Chief Engineer in the development of the plans for the Golden Gate Bridge, and being fully familiar with the initial plans and the report on the same submitted to you by the Chief Engineer, we beg to report as follows:

Immediately after our appointment and from the beginning of the development of the present plans, we have familiarized ourselves with the essential data and conditions bearing upon the project and particularly with all factors which influenced the design of the main span. We have inspected the site of the bridge, its approaches and highway connections on both sides of the bay; have made a thorough examination of the borings and other information relating to the subsurface conditions, and have conferred with the Consulting Geologist relative to the geological structure.

Jointly with the Chief Engineer and his staff, we have determined all the basic loads and forces, the permissible unit stresses and all other provisions of the design specifications which are essential for the safe proportioning of the structure and in conformity with the best practice applied to other large modern structures of this kind. The general type of the main structure and of the approaches, and their proportions and principal details were similarly fixed to conform to economy and good appearance after consideration had been given to other possible types and arrangements.

It was the desire of the Chief Engineer to give especial attention to the architectural treatment befitting the dignity of the structure, and we have been in thorough agreement with him in that respect, and feel that the bridge as designed will present a pleasing appearance.

We have passed upon the initial detail plans which have been elaborated with great care, and with unusual thoroughness, by the staff of the Chief Engineer, and have assured ourselves that they conform to the specifications and to best modern practice.

While, as is always the case in a structure of this kind, the entire design and all details will be gone over and checked repeatedly before the award of the construction contracts and during their execution, we have thoroughly approved of the desire of your Honorable Board and of the Chief Engineer, of having the design computations and details checked independently at the present time, principally to assure the sufficiency of the estimated quantities and costs.

This independent check has been made by experienced engineers under the direction and the immediate supervision of Mr. Leon S. Moisseiff, Consulting Engineer.

The prices used in arriving at the estimated cost have been fixed in conference with the Chief Engineer and his staff. In doing so we have been guided largely by the prices experienced in the case of the Hudson River Bridge in New York, a closely comparable structure.

We have studied the traffic situation and examined the thorough report of the Traffic Engineer, and we consider his findings conservative.

In conclusion and as a result of our close cooperation with the Chief Engineer, and our familiarity with the plans, we are pleased to report, that the design conforms in every respect with the best modern requirements; the stress computations are correct; the sizes and proportions sufficient, and the estimated quantities and costs adequate. We have, as stated, verified the Chief Engineer's estimate of construction and operating cost and are firmly convinced that the bridge can be built and opened to traffic within the estimate of $33,000,000.00, carrying charges included; that it can be operated and maintained for $250,000.00 per annum the first year, the amount increasing in subsequent years to a maximum of $350,000.00, and that the revenues will be sufficient to cover fixed and operating charges and leave a balance at the close of the first year of operation and retire the whole bond issue at the bond maturities.

Respectfully submitted,

/s/ O. H. Ammann,
Consulting Engineer.

/s/ C. Derleth, Jr.,
Consulting Engineer.

/s/ Leon S. Moisseiff,
Consulting Engineer.

REPORT ON CORRELATION OF SUBMARINE TOPOGRAPHY WITH BORDERING GEOLOGIC FORMATION

April 4, 1958

Board of Directors
Golden Gate Bridge and Highway District
San Francisco, California

Attention: Mr. James Adam, General Manager

Gentlemen:

I am pleased to transmit herewith a report relative to the correlation of the latest submarine topography of the Golden Gate with known geological formations of the bordering land.

The accompanying submarine topography (Figure No. 1), based on the latest hydrography by the Coast and Geodetic Survey, has been compiled at a scale of 1-20,000 from two survey sheets. The survey to the west of the Golden Gate Bridge was made in 1954 at a scale of 1-20,000 whereas the survey to the east made in 1948 was on a scale of 1-10,000.

A copy of another special survey made in 1948 at a scale of 1-2400 beneath the bridge alignment will be furnished later for your files.

Very truly yours,

/s/ W. M. Gibson

W. M. Gibson
Rear Admiral (Retired)
U. S. Coast and Geodetic Survey

WMG:mam
Enc.

101 Wildwood Ave.
Piedmont, California

CORRELATION OF THE SUBMARINE TOPOGRAPHY OF THE GOLDEN GATE WITH THE ADJOINING SHORES

April 4, 1958

The northwesterly trend of the ridge of the Coastal Range along the San Francisco Peninsula is interrupted by the Golden Gate, and at intervals with other transversely trending ridges and valleys. The high land to the north of the Golden Gate trends transversely from Point Bonita to Sausalito. A broad valley extending eastward from Rodeo Cove cuts deeply into the main northwest ridge just south of Sausalito and almost joins the headwaters of the southerly trending Horseshoe Bay Valley.

Drainage patterns are not conspicuous in the transverse ridge bordering the Golden Gate to the north, except for the broad valley reaching the Golden Gate just west of the Golden Gate Bridge in the Fort Baker Military Reservation. (See U. S. G. S. San Francisco and Vicinity Map - 1954.) This southerly valley does not have a submarine extension.

The Horseshoe Bay valley does continue to the southward beneath the waters of the San Francisco Bay (Figure No. 1). The west bank of this valley is the downsloping ridge at Lime Point. The east bank is the ridge containing Cavallo Point and extending southward beneath the water beyond the middle of the Golden Gate. This submerged southern extremity of Cavallo Point ridge is fringed by a moat-like depression (which is all that separates it from the San Francisco formation descending from Fort Point to the south). The "moat," in reality, is the bottom of another trough to the eastward emanating from Richardson Bay.

The submerged southern tip of the Cavallo Point Ridge is described on Figure No. 1 by the 210-foot depth curve. The trough emanating from Richardson Bay may best be visualized by observing the long narrow bight of the 180 foot depth curve extending to the northward.

Stated in another way, it may be said that the submarine extensions of Richardson and Horseshoe Bays fringe the submarine extension of Cavallo Point on the east and west, respectively, and form a junction in a narrow elongated depression near the foot of the slope from Fort Point and about 3,200 feet northeast of Fort Point. However, the submarine extension of the Lime Point ridge forming the west bank of the Horseshoe Bay submarine valley fades out at the foot of the scarp, thus presenting a wide opening to the westward.

Assuming that sea level were 210 feet lower in past geological ages, or that the sea floor were 210 feet higher with respect to sea level, the Golden Gate would have comprised a narrow gorge about 450 feet wide and 30-40 feet deep through which drainage from the north joined the sea.

Surface drainage features of the San Francisco Peninsula have no submarine counterparts with the exception of Lobos Creek. However, as will be seen later, the submarine extension may apply more to an ancient bedrock stream rather than to Lobos Creek. A sheared zone along the shore at Lands End probably extends to the northwest to encompass Mile Rock.

The serpentine ridge that includes Fort Point forms a symmetrical bulge sloping down beneath the water and extending out to the middle of the Golden Gate. This symmetrical bulge is fringed at its base by an east-west elongated depression reminiscent of current scour by the streams from the vicinity of Horseshoe and Richardson Bays. In the middle of the elongated depression and lying partly beneath the Golden Gate Bridge, there is a dome-like formation rising about 18 feet above the sea floor. It extends for about 1,000 feet in an east-west direction and about 600 feet north-south.

The significance of this formation is not known, but it evidently lies over the Marin Sandstone dome shown in the geological cross-section of the Golden Gate Bridge alignment beneath the strata of Ingleside chart (Page 18, Golden Gate Bridge - January 1938).

Julius Schlocker, D. H. Radbruch and M. G. Bonnilla, of the U. S. Geological Survey, have compiled a preliminary map showing the shape of the bedrock surface in San Francisco. It is on open file. The areas where bedrock is exposed at the surface are outlined on the map, and the sub-surface contours show the approximate position and shape of the bedrock surface in areas where it is buried by clay, silt, sand and other younger deposits.

Near the Ferry Building bedrock lies nearly 300 feet below the present surface, yet is exposed at the surface a few blocks to the northwest and southeast. This is believed to represent a stream valley carved in the bedrock in past geologic ages during a period of lowered sea level. The buried stream valley is believed to have extended from the Twin Peaks area to the site of the Ferry Building and northward to a junction with the ancient Sacramento River.

This example is beyond the limits of Figure No. 1, but the axes of similar valleys carved in the bedrock have been sketched on Figure No. 1 from the U. S. G. S. map. One of these extends northwest to Bakers Beach near the mouth of Lobos Creek. At the beach, bedrock is 25 feet below sea level, but to the northeast and southwest it is 300 feet above sea level. This constituted a sizable stream flowing into the area of the present Golden Gate. The southward bends in the depth curve between and including the 60 and 270-foot curves confirm its former location now lying beneath the waters of the Golden Gate.

A similar bedrock stream from the south crosses the present shoreline at Crissy Field only 1,800 feet south of the long east-west trough cutting Presidio Shoal. This bedrock stream and another one crossing under the present shoreline at Fort Mason appear to be related to the large trough or basin flanking the south edge of Presidio Shoal and Alcatraz Island.

In general, the ridges and valleys of the sea floor preserve their forms with considerable clarity, in contrast to the land forms, because of their comparative immunity to the great weathering and erosive actions on the land above the sea. In this case, the great amount of sediments carried through the Golden Gate from the Sacramento and San Joaquin Valleys could only be deposited by counter currents along the edges of the main streams and on the delta outside the Heads.

Removal by the currents of the lighter materials would be expected in the constricted channels, such as Raccoon Strait and the Golden Gate. The coarse gravel and pebbles would move more slowly or remain in place. Figure No. 2 contains the sketched locations of the gravels at the time of the 1859 charting; Figure No. 3 is a generalized sketch of the directions of the main currents today and may be assumed to hold good for the period of recorded history in California.

It will be seen that the gravels are found only where strong currents flow, and mud and sand found in areas of weaker currents and counter-currents, such as Richardson Bay, Bonita Cove, South Bay, and portions of San Francisco Bay.

Presidio Shoal is probably a sandstone formation similar to Alcatraz Island. In 1859 it came within 4 or 5 fathoms of the surface. Shag Rock was 4 feet and Arch Rock 35 feet above sea level. Presidio Shoal and the two rocks have been lowered by blasting and dredging. The transverse orientation of Presidio Shoal may be explained as the south bank of the ancient Sacramento River or as a drowned portion of the San Francisco Peninsula.

The basins off Point Bonita and Mile Rock are believed to have resulted from the removal of overburden by the turbulence created by the junctions of the main and counter currents. During certain stages of the current cycle, the direction of flow in Bonita Cove and South Bay is contrary to the main flow in the Golden Gate. This creates a great deal of turbulence in the water off Pt. Bonita and Mile Rocks and it is here that deep basins have been scoured. Similar basins have been scoured below the north shore of Angel Island, where the current from the north impinges directly upon the Island base and is diverted radically to the east and west.

A special study of the coast at Lands End by Schlocker, Bonnilla and Imlay of the U.S. Geological Survey (Bull. of Am. Assoc. of Pet. Geol., v. 38, p. 11, Nov. 1954) describes abundant shearing, faulting and jointing in a sequence of sediments dipping eastward at about 30 degrees. The numerous faults in the exposures of this small zone are interpreted by the authors as minor breaks and readjustments since the deposition. There is no evidence there of a northward trending fault.

It is believed that underwater faults formerly subjected to differential vertical or horizontal movement would be indicated by offsets in the depth curves. No such offsets are noted. Rough comparisons between the recent submarine topography and that of earlier surveys going back as much as 100 years do not show really substantial changes. Although differences are noted here and there, including depths in the middle of the Golden Gate, they may be largely ascribed to the difficulty in the early days in getting good soundings and positions in the strong currents. The old depths were greater, as would be expected.

CONCLUSIONS

1. The distance between Fort Point on the south side of the channel near the south tower of the Golden Gate Bridge, and Line Point near the north tower of the bridge, is essentially the same now as in 1859, indicating no widening of the Golden Gate through sliding, scouring or earthquake faulting.

2. Most of the submarine forms in the Golden Gate are interpreted to be old formations (at or near bedrock) that have been preserved with unusual clarity by action of water currents, and were not primarily the result of current action. Two sub-aerial valleys and ridges from the north have submarine extensions in the Golden Gate area.

3. Ancient bedrock streams of the San Francisco area correlate reasonably well with submarine troughs. Neither the submarine troughs, their surface, or bedrock counterparts may reasonably be interpreted as faults. There are no submarine formations that may be construed as confirmation of the submarine faults postulated by Bailey Willis in 1934.

4. Basins off Point Bonita and Mile Rock are due to turbulence of water currents which has removed the overburden.

5. The great strength of ebb current has prevented deposition in the Golden Gate and probably removed some of the overburden.

6. Examination of recorded data on the submarine topography of the Golden Gate channel indicates that there has been no slumping, sliding or faulting. Absence of such evidence tends to confirm the stability of the Golden Gate Bridge towers, piers and foundations.

W. M. Gibson, R. Adm. C & GS (Ret)

4/7/58

REPORT ON GEOLOGY AND SEISMOLOGY

SUPPLEMENTAL REPORT

April 4, 1958

Mr. James Adam, General Manager
Golden Gate Bridge and Highway District
Box 99, Presidio Station
San Francisco, California

Dear Mr. Adam:

In accordance with your request under conditions stated in your letter of 27 March 1958, we have pleasure in forwarding herewith a supplemental opinion on the possible effects of Earthquakes upon the Golden Gate Bridge. This is accompanied by a map showing locations of earthquake epicenters 1930 to 22 March 1957.

Outside the scope of this report, we also make two suggestions aimed at collecting future data regarding the stability of the Bridge:

(1) A series of bench marks should be established on the bridge and adjoining bed-rock in the vicinity of both the north and the south foundations. These should be set up, with a view to detecting and measuring any permanent displacement between bridge and bed-rock or between different sections of the bridge that might accompany future earthquakes or other violent natural occurrences. Annual surveys of such points, conducted over a period of years, would yield valuable data on the stability of the Bridge and of its foundations.

(2) A series of photographs of the rock slopes in the vicinity of the foundations should be taken periodically from fixed points. This would yield information regarding the general stability of rock slopes with respect to superficial sliding.

Yours sincerely,

/s/ Francis J. Turner

Francis J. Turner,

/s/ Perry Byerly

Perry Byerly

SUPPLEMENTARY OPINION ON POSSIBLE EFFECTS OF EARTHQUAKES UPON THE GOLDEN GATE BRIDGE

April 4, 1958

On 20 June 1955, the two undersigned together with Professor Charles M. Gilbert wrote jointly "An Opinion on Possible Effects of Earthquakes Upon the Golden Gate Bridge." In March and April 1958, at the invitation of the General Manager of the Golden Gate Bridge and Highway District, we have re-appraised the situation and now submit a supplemental report on the same topic.

This is based upon (1) re-examination of rock formations and surface outcrops adjoining the north and the south foundations of the Bridge; (2) scrutiny of seismograph records of earthquakes taken at the University of California Seismographic Station between June 1955 and 22 March 1957 (the date of the last major earthquake in this region); (3) scrutiny of records of the earthquake of 22 March 1957 taken on the Bridge seismographs; (4) perusal of reports relating to the reaction of the Bridge to the earthquake of 22 March 1957; (5) perusal of a report on soundings in the vicinity of the North Pier, carried out in 1956 by the U.S. Coast and Geodetic Survey.

Accompanying the statement of June 1955 we submitted a map showing all accurately located epicenters of earthquakes originating in the general vicinity of San Francisco over the period 1930-1935. A second map of a similar nature is now submitted, including more recent as well as the original data. This shows all accurately located epicenters of earthquakes recorded from 1930 up to (and including) the earthquake of 22 March 1957. Epicenters of aftershocks following the latter earthquake are not shown; but none of these was closer to the Golden Gate Bridge than those indicated on the map. The revised map completely accords with our previous findings. Again we state that the seismological record offers no indication of an active fault nearer the Bridge than the San Andreas zone.

The seismograph records made at Bridge Stations at the time of the earthquake of 22 March 1957 have been examined. We verify the statements made by Clifford E. Paine in the second and third paragraphs of his letter to the General Manager, dated 2 April 1957. The double amplitudes there recorded (and verified by us) - 5.6 inches and 2.9 inches - are trivial displacements compared with those to which the Bridge is repeatedly subjected for much longer times under heavy gales of wind. This trivial reaction of the Bridge to an earthquake of moderately great intensity accords with our previously stated opinion.

Re-examination of surface outcrops of rock in the vicinity of north and south foundations of the Bridge has yielded no new evidence bearing on the problem. Rock slopes adjoining the foundations appear stable. There is superficial minor sliding of surface rock in road cuts on the road leading to the southern foundation; but these are surface effects due to action of rainwater and have no bearing on the stability of the basement rock mass. Our opinion of June 1955 stands without modification.

New soundings near the North Pier undertaken by the U. S. Coast and Geodetic Survey in 1956 are completely in accord with our previous findings.

All the conclusions stated in our report of 20 June 1955 remain valid. Indeed, they are confirmed by subsequent events. Since June 1955 the San Francisco region has, in accordance with our expectation, been affected by an earthquake of moderately great intensity. Also in accordance with our expectation, the epicenter of this earthquake was located on one of the two principal active faults of the region - the San Andreas zone. In conformity with prediction, the direct effect of the earthquake in setting up vibrations in the Bridge was trivial; and there is no evidence whatever that the earthquake was accompanied by sliding or by other displacement of basement rock in the vicinity of the Bridge foundations.

We re-endorse and confirm our report of 20 June 1955 in every detail, without revision, modification of qualification of any kind.

/s/ Francis J. Turner

/s/ Perry Byerly

7 April 1958

AN OPINION ON POSSIBLE EFFECTS OF EARTHQUAKES ON GOLDEN GATE BRIDGE

Perry Byerly, Seismologist,)
Charles M. Gilbert, Geologist,)
Francis J. Turner, Geologist,)
University of California
Berkeley, California
June 20, 1955

SCOPE AND BASIS OF REPORT

This report embodies a joint opinion of the three undersigned, advising the General Manager and the Chief Engineer, Golden Gate Bridge and Highway District, on geological aspects of certain problems relating to the Bridge foundations. It is concerned with three questions:

1. In what ways might the foundations of the Bridge be affected by earthquake shock and fault movements?
2. Which of these effects would be most likely in the event of a severe earthquake in the general vicinity of San Francisco Bay?
3. Is the Bridge specially vulnerable to such effects as compared with other large structures in the same broad region?

The report is based upon:

1. The writers' general experience in geology and seismology.
2. Published data relating to the geology and seismic history of the region around the Golden Gate.
3. Field inspection of rock formations now exposed above low-tide level in the immediate vicinity of both ends of the Bridge.
4. Off-shore soundings adjacent to both the north and the south pier, taken by the Golden Gate Bridge and Highway District periodically between 1930 and 1940, and by the U. S. Coast and Geodetic Survey in 1948.
5. Information supplied by the Chief Engineer as to construction details, foundations at the time of construction, levels, behavior of the Bridge since construction; copies of previous reports by consulting geologists and engineers.

POSSIBLE EFFECTS OF EARTHQUAKES

An earthquake is a transient vibration of some portion of the earth's crust. It originates from a sudden mechanical disturbance - generally supposed to be displacement on a fault - at a point termed the focus. This is commonly located within 15 miles of the surface. The surface point immediately above the focus is termed the epicenter; and it is this, not the focus, that is determined from seismograph records. A severe earthquake usually lasts less than a minute (the duration of the 1906 San Francisco earthquake was estimated as between 40 and 60 seconds).

From 1930 till the present time continuous seismograph records have been taken at many stations in California, so that the seismic history of the State over that period is particularly well known. Also, there is a considerable fund of information regarding behavior of engineering structures in this region.

Experience shows that in general there are three possible effects of earthquakes in relation to large structures, such as buildings, bridges, dams and highways:

1. Relative displacement of two sectors of the land surface on opposite sides of an active fault passing directly beneath the structure.

2. Destruction by shaking - the direct result of the earthquake vibration within the structure.

3. Secondary local displacements of soil and rock at and near the surface under the structure, caused by the earthquake vibrations. To this category belong landslides, submarine slides, surface cracking, and creep.

SUMMARY OF CONCLUSIONS REGARDING GOLDEN GATE BRIDGE

We concur in the following conclusions with respect to the special problem of Golden Gate Bridge:

A - In accordance with past history, we assume that during the next century the San Francisco Bay region will repeatedly be affected by earthquakes, and that a few of these will be severe. We believe, however, that well designed and properly built structures founded on coherent bed-rock, such as Golden Gate Bridge, will withstand these shocks with minimum damage. We conjecture that the Bridge, had it existed in 1906, would have survived the earthquake of that year without substantial damage.

B - The possibility of destructive effect of earthquakes applies to any large structure or engineering work in the Bay area. We consider the Golden Gate Bridge neither more nor less vulnerable to damage or destruction by future earthquakes than other large structures situated in the Bay region or in regions of comparable seismic activity.

C - In the event of severe earthquake, the Bridge may possibly be affected in the following ways, listed in order of decreasing probability:

1. Sliding and cracking of foundation rocks.

2. Damage resulting directly from vibration of the structure.

3. Relative displacement of north and south piers by movement on a fault (the existence of which is still unsuspected) in the Golden Gate Channel under the center span of the Bridge.

4. Movement on faults (the existence of which is not suspected) immediately under either pier or anchorage.

We consider very remote the chance of fault displacements under the Bridge (3 and 4 above). The chance of damage from vibration (2) is probably remote but depends more on design and construction than on geological conditions. The most likely damaging effect is surface sliding of the rock foundation (1), but the character of the area and its behavior during the 20 years since construction suggest that this possibility, too, is unlikely.

In the following pages we have detailed and explained the basis of the conclusions summarized above.

POSSIBLE DISPLACEMENT ALONG ACTIVE FAULTS

General Statement

One of the most spectacular effects of an earthquake is relative displacement of portions of the land surface on opposite sides of an active fault. At the time of the 1906 earthquake, for example, lateral displacements locally reaching a maximum of 20 feet occurred along the San Andreas Fault for a distance of over 200 miles. The Nevada earthquake of December 16, 1954, was accompanied by the development of vertical scarps, locally as much as 23 feet high, along several faults in an area 50 miles long. However, most earthquakes are not accompanied by surface breaks of this kind, even though the epicenters may be situated on or close to known fault traces. When surface breaks do appear, they commonly, but not invariably, occur along pre-existing faults.

To understand these facts, one must understand what a geologist recognizes as a "fault." To him a fault is a surface or zone in the earth's crust along which relative movement of the two sides has occurred at some past time. As revealed by surface mapping, the geologist recognizes and delineates a fault by such criteria as abrupt discontinuity in the geologic formations, zones of sheared and pulverized rock, striated rock surfaces (slickensides), and sharp linear breaks in topography. But the designation and delineation of a fault is not a prediction that future displacement will occur along it. Current and probable future activity along a fault is surely indicated only by surface displacements within historic times or by local clustering of epicenters of recorded earthquakes. Thus the existence of a fault on a geologic map does not necessarily imply present or future instability of the land surface at its outcrop, though it undoubtedly weakens the rock for foundation purposes.

The basement rocks of the San Francisco Bay region, on which the Bridge is founded, belong to the Franciscan formation formed some 100 million years ago. Since then, these rocks have been repeatedly folded and faulted, so that hundreds of faults have been demonstrated in the Franciscan rocks of the central California Coast Ranges. On only a very few of these have surface displacements been recorded in historic times. There is no reason to believe that movement has occurred within the past half million years on any of the great majority of known minor faults cutting the Franciscan rocks of California.

Among recently active faults in the Bay area, the two best known are the San Andreas (active in 1906) which passes within 6 miles of the Golden Gate, and the Haywards Fault (where cracking occurred in 1868) some 12 miles to the east, along the Berkeley Hills.

Great damage might result from large surface displacements along faults passing under the center span of the Golden Gate Bridge or any of its foundations if any such faults exist. Although we regard this possibility as remote, it is pertinent to search for evidence of possible existence of active faults so located.

The Golden Gate Bridge Site

A - Evidence from Earthquake Epicenters. The accompanying map shows epicenters located within 20 miles of Fort Point during the period 1930-1954. There are plotted here only the epicenters which were considered as well located. They are taken from the Bulletin of the Seismographic Stations of the University of California. These epicenters are rated a, b, c, d, according to the decreasing accuracy of the location. Since c and d epicenters may be some tens of miles in error, there are plotted on this map only those of rating a and b. In the Geologic Guidebook of the San Francisco Bay Counties, Byerly presented a map showing epicenters for a more restricted period. On this map some epicenters not appearing on the enclosed map. These were of rating c and d. However, none of them was nearer the Golden Gate Bridge than those appearing on the accompanying map.

It can be seen that the seismological record offers no indication of an active fault nearer the Bridge than the San Andreas zone. Regarding the epicenters outside the Golden Gate, it will be noted that all do not lie exactly on the line drawn to represent the fault. This is also the case on land to the north and to the south. In our opinion these are all attributable to that fault. The scatter may be due to width of the fault zone, but is more probably due to errors in locating the epicenters caused by slightly inaccurate knowledge of the velocities of seismic wave.

B - Evidence from Surface Mapping. Rocks in the vicinity of the northern foundations are varied in character (basalt, chert, and sandstone) and well exposed. If major faults exist here, there should be clear evidence of their existence. None was mentioned in earlier reports, and we have found none. To be sure, numerous slickensided surfaces and seams of sheared rock are to be seen in the cliffs along the shore, but such surfaces and seams do not interrupt the continuity of the formations and cannot be classed as faults of any significance from a seismic standpoint.

At the southern foundations, the bedrock is serpentine. It is here impossible for anyone to prove by geologic mapping whether or not this serpentine mass is crossed by important faults.

The possible existence of a fault boundary between sandstone and serpentine along the shore west of the Presidio and south of the south anchorage remains an open question. Professor A. C. Lawson, one of the consulting geologists for the Bridge District, contended that this contact is not a fault. Professor Bailey Willis disagreed, and described the contact as a fault dipping at a low angle toward the east so that sandstone would underlie serpentine at shallow depth beneath the south pier (Argonaut, Oct. 19, 1934). To determine the accuracy of Willis' picture, a boring at the west end of the south pier was drilled about 200 feet below sea bottom and 159.37 feet below the base of the pier. Only serpentine was encountered and Willis' contention was considered to be completely disproved. It must be emphasized, however, that this one boring proves only that his projection of bedrock structure beneath the pier was quantitatively in error; the boring by no means disproves the existence of fault a few hundred feet west of the pier.

The second geological consultant for the Bridge District, Mr. A. E. Sedgwick, lends some general support to the idea that a fault exists in this general position. He states in his report of February 1931, entitled Foundations of the Golden Gate Bridge (page 12):

> "No evidence was found that faults run through the pier sites. The active San Andreas fault is about six miles west. There is some evidence that two other faults lie between the bridge site and the San Andreas fault. One of these would pass but a few hundred feet west of the south pier. There is some evidence that it has been active recently. Great caution, therefore, should be taken in the bridge design to resist the stresses originating from seismic disturbance."

For completeness, it must be added that Sedgwick did not again refer to this suspected fault in any of his later statements to the Bridge District.

Any field evidence for the existence of this hypothetical fault must be sought in the exposed cliffs south of the south anchorage. In our opinion, the bed-rock along these cliffs is so disturbed by superficial sliding (discussed later) that nearly all observations are somewhat questionable. There is indication of a severely sheared contact, between serpentine to the east and sandstone to the west, which trends slightly west of north. Thus, our observations tend more to support Willis and Sedgwick than Lawson. But we do not hesitate to say that no geologist should make an unequivocal statement on this matter without the evidence from a planned drilling program. In any case, no historic displacements have been recorded along the suspected fault line, and no epicenters have been located there.

The existence of a fault near the south anchorage is questionable. If it does exist, its location relative to the south anchorage and the south pier is unknown; and there is no indication of any kind pointing to displacement on such a fault within historic times.

C - The Evidence from Topography. The Golden Gate Channel is a submerged east-west trending stream valley. In California and elsewhere, not a few streams have eroded their valleys in the shattered rock along faults and therefore tend to follow these faults for considerable distances. It is a common possibility, therefore, that a bridge spanning a river valley, or a drowned valley, may also span a fault. In the California Coast Ranges this possibility applies particularly to valleys trending northerly or northwesterly, parallel to the regional structural trend in the basement rocks. There is no positive evidence whatsoever that the Golden Gate Channel is located along a fault zone, and its westerly trend makes the possibility unlikely.

Topographic details of the landscape adjacent to the two ends of the Bridge provide no indication of recent faulting there. To be sure, as evidence of recent faulting in the immediate vicinity of the south pier, Professor Bailey Willis reported alleged changes in the submarine topography. The alleged changes were based on a comparison of soundings made in the area by the U. S. Coast and Geodetic Survey in 1895 and again in 1920. The reality of this evidence is extremely questionable and is rendered untenable in the light of more recent soundings. It will be considered further in connection with the possibility of submaring sliding.

D - Conclusions. We conclude that:

1. No active fault or faults have been proved to exist beneath any of the Bridge foundations nor in the Golden Gate Channel; we find no positive evidence that such a fault may exist.

2. Continuous records over 25 years (since establishment of sufficient seismograph stations for complete and accurate location of epicenters) show that over this period the Golden Gate has not been a center of seismic activity. The less reliable earlier records give no indication of such activity in the vicinity of the Golden Gate prior to 1930.

3. As for any other locality in the Coast Ranges of California, a surface break in the immediate vicinity of Golden Gate Bridge is a possibility in the event of future earthquake. This possibility appears no greater for the Golden Gate than for other bridge sites in California.

4. The chance of surface break on a fault at a particular locality is remote compared with other destructive effects of earthquake shock to be considered in the remainder of this report.

POSSIBLE DIRECT EFFECTS OF VIBRATION

Within historic times five earthquakes of great destructive intensity have affected the San Francisco Bay region. It is certain that equally intense disturbances will recur in the future. Whether the epicenter of a future severe earthquake should be located on the San Andreas or Haywards Fault or elsewhere in the general vicinity of the Bay is immaterial to this discussion. The important variables that will determine which structures will survive and which will be destroyed are (1) construction and (2) the rock in the foundations. Within a radius of a dozen miles, proximity to the epicenter is of much less significance than either of these factors. There is no record that any building or engineering work, properly designed and constructed with a view to resisting earthquake shock, has ever been destroyed by direct effect of earthquake vibration in California.

Experience in California, Japan, New Zealand and other seismically active regions shows that structures resting on bed-rock are very much less vulnerable to destruction by vibration than are structures built on watersaturated valley fills or reclaimed land. In this respect Golden Gate Bridge is well located. Its foundations, including both piers and anchorages, are sunk deep into bed-rock of the Franciscan formation. It is thus decidedly less vulnerable to vibration than are bridges and buildings built on sedimentary fills (river gravels, reclaimed land, etc.).

This generalization is borne out by reference to the map (and accompanying cross-sections) showing distribution of apparent intensity of the 1906 earthquake vibrations in the San Francisco area (Plate 19 of Atlas published by State Earthquake Investigation Commission, California Earthquake of April 18, 1906). On an intensity scale decreasing from a to e, Fort Point is shown as part of a large area where the apparent intensity ranked d. Much of this area is underlain by serpentine. Indeed, comparison of the earthquake-intensity map for the 1906 shock and a geological map of the same area shows that in large areas of serpentine the intensity of vibration was generally low.

We conclude that the nature of the rock on which the foundations of Golden Gate Bridge rest favors its resistance to direct effects of vibration in future severe earthquakes.

POSSIBLE SUPERFICIAL DISPLACEMENTS ON SURFACE ROCK

General Nature of Such Displacements

Various kinds of superficial displacements are commonly recorded in seismic areas; but of these only downslope creep or sliding, and accompanying cracking of the surface, concern us here. By sliding we mean the downslope movement at perceptible rate under gravity, of surface masses of bed-rock and soil cover; creep is similar except that the rate of movement is imperceptible except over a number of years. Such creep and slide masses may be a few square feet or many acres in extent, and their vertical displacement may be a few feet or hundreds of feet depending upon the details of slope down which they move. The sole, or bottom surface, of the slide mass may be at the solid bed-rock surface or may cut many feet into bed-rock; but from the geological point of view, in contrast to faulting, slides are superficial phenomena.

Creep and many slides often occur without any relation to earthquakes. However, in a seismic region such as the Bay area, they are commonly induced by earthquake vibrations in places where the surface rock is already unstable. They are, therefore, partly the result, not the cause, of earthquakes, and they are liable to occur anywhere throughout the area affected by an earthquake shock. Indeed, they usually cause much more damage to man-made structures than do the more spectacular but localized surface displacements along active faults.

At any given point in a seismic area, the stability of the surface rock material (bedrock and soil cover), and therefore the susceptibility to downslope movement, is determined by the character of the rock, the steepness and length of slope, and the moisture content of the rock and soil. Proximity to earthquake epicenters is generally less significant. If other things are equal, however, the greater the local intensity of vibration, the greater will be the chance of local sliding.

Certain types of rock are more likely to slide than others. Massive well-cemented sandstones of the Franciscan type, and volcanic rocks such as diabase or basalt are on the whole resistant to sliding. Thinly-bedded chert is more prone to slide. So also are loosely cemented sandstones and mudstones and some kinds of serpentine. Apart from its lithological nature as just discussed, the structure of a rock mass greatly affects its potentiality to slide. Where bedding surfaces or shear surfaces within the mass dip in the same general direction as the slope of the land surface, sliding is much more likely than when such structural surfaces dip inward with respect to hill slopes.

Though it is impossible accurately to estimate the probability of sliding on the basis of the three variable factors mentioned above, fortunately predictions for a given locality can be made on the basis of previous behavior of surface rock at the locality in question. Sliding is a surface phenomenon. It tends to recur repeatedly and frequently at a locality where topography, lithology and rock structure favor local surface movements. At such a locality surface cracks and other evidence of minor sliding are obvious; cracks repeatedly develop in building foundations and in walls of masonry. Absence of such signs of instability in any area that has been under close observation for a few tens of years suggests that future sliding or creep are unlikely. This negative evidence is especially cogent in regions frequently affected by earthquake shock.

The Golden Gate Bridge Site

A - Evidence from Submarine Slopes. Soundings indicate that there is a considerable slope of the sea bottom from either Bridge pier toward the Golden Gate Channel. From the north pier the bottom slopes uniformly south at an average angle of 11°, and the pier is about 300 feet above the channel floor. From the south pier the slope is steeper and less uniform; the base of the pier is about 215 feet above the channel floor. Under the center line of the bridge the slope from the south pier averages 15°; the slope in a northeasterly direction averages about 20°; that in an easterly direction, about 13°.

Thus the submarine topography of the Golden Gate is such that sliding (of a type which might affect the Bridge) is a possibility to be considered. Mr. A. E. Sedgwick considered the facts, and from his analysis of the stresses involved, he concluded that there was no danger of sliding toward the channel caused by the load on the pier foundations (Report on the Foundations of the Golden Gate Bridge, February 1931, pp. 9-11). On the other hand, Professor Bailey Willis compared soundings taken by the U. S. Coast and Geodetic Survey in 1895 and 1920 and concluded that slide and possibly fault movements had occurred during that interval on the slope north of the south pier. The evidence on which this alarming conclusion was reached is not real. Even the Director of the Coast and Geodetic Survey objected to the use of these old soundings for purposes of precise comparison (Report of the Building Committee, Golden Gate Bridge and Highway District, Nov. 27, 1934, pp. 8-9).

Soundings taken by the Bridge District in 1934 and again in 1939 and 1940 are probably as accurate as any yet recorded. Duplicate soundings at precisely the same point are not possible and hence the series of soundings are not strictly comparable; but no general change during this interval is apparent from scrutiny of all available soundings.

Comparison of soundings taken by Bridge District engineers between 1930 and 1940 with extensive soundings made in 1948 by the U. S. Coast and Geodetic Survey indicates a general shallowing of water in the Golden Gate. Around the south pier the apparent differences are as much as 20 feet in some places and topographic features appear altered. Dames and Moore record the same general change on the slope south of the Marin pier (Foundation Conditions of the Marin Pier, Dec. 6, 1949, pp. 13-14 and Plate 5). There is, however, conclusive evidence that bench marks located on shore, and also the elevation of the piers, have not altered appreciably during this time.

These facts are open to two alternative interpretations:

1. There is too much error in some or all of the soundings for these to be used for comparative purposes to estimate changes in submarine topography; or

2. The bottom in this area is mantled with loose rubble and sediment which is moved by currents from time to time.

If this latter is the actual condition, then clearly changes in bottom level and contour are unrelated to, and afford no evidence of, movement in bedrock on which the piers are based. In either case soundings must be completely discarded as evidence of change in the bedrock, and Professor Willis' contentions regarding bedrock sliding and possible recent activity on faults at the south pier has no factual basis.

B - Evidence from Bedrock. According to previous geologists' reports (e.g. Sedgwick, February 1931, p. 12), the rock immediately underlying the north pier is diabase and chert, which itself rests on massive cemented sandstone. These rocks are of the type generally considered as unlikely to slide.

In previous reports attention, very properly, has been focused on the possibility of sliding beneath or near the south pier. The basement rock here is serpentine. And in the California Coast Ranges some types of serpentine are undoubtedly prone to surface sliding. This is true more particularly of two kinds of serpentine:

1. Highly sheared, loosely compacted masses of serpentine rubble squeezed up along active faults. Some of the well known slide areas in the East Bay are underlain by loose serpentine rubble of this type, squeezed up along the active Haywards Fault.

2. Sedimentary serpentine - a porous superficial mass of detrital fragments of serpentine derived by erosion or sliding from pre-existing primary serpentine masses.

The serpentine mass at the southern foundations of Golden Gate Bridge belongs to neither of the types listed as 1 and 2 above. It is a primary instrusive mass. Though sheared throughout its whole extent, it is a coherent mass. The shearing is believed to have occurred at the time of intrusion, under very great confining pressure. Consequently the mass is tightly compacted and non-porous. Its low permeability to water is proved by the fact that the excavation for the south anchor block, carried to a depth of many feet below sea level, remained dry in spite of its close proximity to the sea. The dominant internal structural feature of the South Foundation serpentine as exposed today near the anchorage is the presence of innumerable shear surfaces which on the whole dip landward. This attitude of the shear planes renders sliding less likely than if the shear surfaces prevailing dipped seaward. We have no information regarding dip of shear surfaces in serpentine immediately beneath the pier itself.

In our opinion, serpentine is a rock more prone to slide than the massive sandstones and diabases of the Franciscan type. The serpentine mass of the South Foundations appears, however, to be one of the more resistant types of serpentine. Further evidence of its stability is the relatively low apparent intensity of vibration (d in a scale decreasing from a to e) recorded for Fort Point and the vicinity at the time of the 1906 earthquake.

C - The Evidence from Topography of the Present Shore. One pertinent fact so obvious that its significance may be overlooked is that the Bridge spans the narrowest part of the Golden Gate, and is anchored on cliffs on either side. Since the channel is a drowned stream valley produced by erosion, the narrow passage and high cliffs suggest that here the rocks are resistant. Fort Point, the south anchorage, projects far into the stream and would not exist had the serpentine mass beneath it been subject to repeated sliding toward the channel.

Along the cliffs west of the Presidio and south of the Bridge, there have been numerous geologically recent slides. These were cited by Professor Willis as evidence that the foundations of the Bridge are unstable (Report of the Building Committee, Golden Gate Bridge and Highway District, Nov. 27, 1934, p. 6). These slides, however, have no relation to the submarine foundations. They occur on a cliff facing the open sea and are the direct consequence of undercutting by storm waves attacking the cliff base. The cause of the slides is wave erosion near sea level, and so their limit in depth is approximately sea level. Along these cliffs, as elsewhere in California, the most impressive slides are in serpentine; but sandstone is also included.

Continued pounding of the waves along an exposed cliff, such as the one west of the Presidio, causes gradual but continuous recession of the cliff. Here, recession is indicated clearly, but its rate is difficult to measure. Along the shore there is evidence that some small man-made structures have been destroyed over the years by the eastward encroachment of the sea. Within a century, very considerable recession of the cliff may occur and the heads of the slides along it will likewise recede. At the south anchorage, protection against progressive erosion has been erected in the form of a sea wall. This protection will one day need to be extended southward, for there is a small slide that heads only 125 feet west of the first roadway support at the south end of the Bridge.

It is emphasized that the advice in the preceding paragraph has no bearing upon the possibility of sliding of deeply submerged foundations such as that beneath the south pier.

D - Recorded Evidence Since the Beginning of Construction. The Chief Engineer and the Superintendent of the Bridge report no sign of settling, tilting, or change in Bridge level or alignment over the period in which the Bridge has been in existence. In our opinion, this is very significant evidence of the stability of the structure and its bedrock foundations. One might have expected, however, that this evidence would be the most precise and complete available, including the data of annual surveys and levelling of established points on the Bridge piers and on both shores. Actually, we have only the qualitative statements of the engineers that no appreciable change has occurred. The most recent survey of the Bridge and foundation site was completed in 1949 under the supervision of John G. Little, Consulting Engineer (now deceased), but no quantitative comparison of these and earlier data is available.

In 1952, the U. S. Coast and Geodetic Survey completed relevelling of five bench marks on and near Fort Point, the south anchorage of the Bridge. These had been previously levelled in 1932. At the present time only preliminary 1952 levels, not adjusted for datum, are available for these points, and they are not to be precisely compared with the standard 1932 levels adjusted to sea level datum of 1929. The figures for three bench marks located on concrete structures in the area of the old fort (Tidal 9, T108, S108) suggest, however, that no significant change in level has occurred in the immediate vicinity of Fort Point during this twenty-year interval.

Our examination of the south pier foundation and anchorage found no visible signs of cracking in the concrete structures and suggests that there has been little if any movement of the bedrock beneath. The 700-foot plumb-line installed in the west tower on the south pier is still in the position expected by the designing engineers. Our observations of the concrete in the north foundations were brief and probably of little value. It should be recorded, however, that Dames and Moore (Report on Foundation Conditions of the Marin Pier, Dec. 6, 1949, p. 15) discovered visible cracks in the southwest corner of the north pier, but only there.

Conclusions regarding Possible Surface Displacements

We conclude that scrutiny of areas adjoining both north and south foundations of the Bridge and of the Bridge itself over a period of more than twenty years fails to reveal signs of surface instability. The observed behavior suggests that both foundation areas are essentially stable and no more likely to slide under the impact of future earthquakes than other areas of comparable relief in the Bay Area.

/s/ Perry Byerly, Seismologist

/s/ Charles M. Gilbert, Geologist

/s/ Francis J. Turner, Geologist

Signed, 20 June 1955

June 23, 1955

Mr. James Adam, General Manager
Golden Gate Bridge and Highway District
Toll Plaza
San Francisco, California

Dear Mr. Adam:

On pages 19 and 20 of our recent report to you, reference is made to levels of bench marks Tidal 9, T108 and S108. Today we have received from the U. S. Coast and Geodetic Survey adjusted elevations for these bench marks based on levels taken in 1932 and again in 1952. These are as follows:

	1932	1952
Tidal 9	199.360 ft.	199.389 ft.
T108	184.882 ft.	184.911 ft.
S108	37.802 ft.	37.802 ft.

As you will see, these figures indicate changes in elevation ranging from zero to approximately one-third inch. These I regard as of no significance.

Yours sincerely,

/s/

F. J. Turner

A RATIONAL BASIS
OF
EARTHQUAKE INSURANCE

Address of
DR. BAILEY WILLIS
President of the Seismological Society of America,
delivered before the Annual Meeting of

THE NATIONAL BOARD OF FIRE UNDERWRITERS

New York, May 24, 1926

Mr. President and members of the National Board of Fire Underwriters: When I was asked to address you on this occasion, I was impressed chiefly with the responsibility attending the opportunity. I do not measure that responsibility by the millions of investment, to which your President referred this morning, seven hundred and seventy millions, nor by the billions of property that you insure; nor, even, by the brains and leadership represented in this assembly. It appears to me to be imposed, because, within my experience, this is the first time that a body of this importance and of this weight, has turned to science and asked for a public statement of what science knows on this particular scientific subject.

You apply scientific methods in your accounting, in your adjustments, to some extent in various other branches of your business, but you rarely go outside of your organization to ask for research and its results as you have in this case.

I speak to you, therefore, as a representative of science, and I wish to make it understood that in so speaking I speak primarily for research, without which we must all of us cease to anticipate the further development of our great civilization. I speak also for applied science, without which our genius would not be fruitful and helpful in promoting that civilization.

The object of this address is to present the data for a reasonable understanding of earthquakes and to deduce a rational basis for insuring the public against losses where they occur. It is necessary first to understand what an earthquake is; how often it is likely to repeat in a century; how it acts; and what it does. Then we may consider how to insure against it in such a manner that the insured.may be protected and the insurer need not run undue risks. What is an earthquake? It is an elastic snap of the earth's crust, just such a snap as you can give to a steel rod if you bend it to one side and let it go suddenly. Rocks under pressure in the earth's crust are as elastic as steel. They are pressed out of shape. They yield like the steel rod by bending and as the pressure acts very slowly they bend for many years before they snap. But when they do snap all the energy that has been stored up in their elastic masses is instantly released and the crust is set to vibrating for many miles around the break or earthquake focus.

The mechanical action is not always so simple. It happens in some districts that during the accumulation of the strain little slips occur. Each of them produces a slight earthquake. When the pressure approaches the breaking point the incidental slips may release more force and we get heavier shocks anticipating the greatest of the series. Then, when it has been sprung, there follow hundreds of after-shocks. This is a common type of earthquake activity, characteristic of regions where destructive effects are likely to be experienced.

In this connection I wish to coin a phrase, namely earthquake habit. The term earthquake habit shall be understood to mean the succession of shocks which is characteristic of a given district; the frequency, intensity, and extent of area disturbed being taken into account in distinguishing different habits.

It will, I think, be evident that a knowledge of the earthquake habit of any district is necessary to any rational insurance policy for that area. To insure the insured and at the same time insure the insurer we must know how often earthquake shocks of some certain degree of intensity may occur and what area they will probably affect. But if we know these things we know the earthquake habit.

I happen to be acquainted, more or less superficially, with the earthquake habits of four different regions. I know the earthquake habits of California fairly well. I know the earthquake habit of Chile, because I have investigated the results of a severe earthquake there, and have studied the records of the last four hundred years. I have something to tell, less than I should have, of the earthquake habit of the Atlantic Coast, and I know the history of earthquakes in Great Britain.

Let me try to distinguish those for you. Chile has about a thousand earthquakes a year, that is, perceptible earthquakes. They are pretty numerous. The Chilean earthquakes, when they reach their highest intensity are violent, but not as extremely violent as certain earthquakes of Japan or India have been. A very severe earthquake occurs about every twenty years while intervening shocks are merely startling. A great Chilean earthquake shakes a very large area, something like three or four hundred thousand square miles. The last one, in November, 1922, shook the coast for fifteen hundred miles in length, that is as far as from Florida to Newfoundland approximately, and not only disturbed islands five hundred miles west of the coast, but extended vigorously to the Atlantic shore. It originated in a centre of great earthquake activity, somewhat deeply located, and released its elastic rebound on a flat surface which is twenty or thirty miles down in the earth, and which passes under the Andes. The effect is that of pushing the whole Andes eastward so that when a slip occurs the Andes advances a little step. Chile is invading Argentina to that extent.

The earthquake habit of California, taking the State as a whole and basing the estimate on the records of the last century and a half, corresponds to one shock of intensity VIII or higher on the Rossi-Forel scale every three or four years. If we restrict the estimate to shocks of greater intensity, say IX to X R.F., for instance, the habit is represented by intervals of about thirty years. These averages present only half the picture, however. The shocks are not so evenly spaced. There will be an interval of twenty years more or less, during which there are practically no disturbances of notable intensity; several startling shocks then follow a few years apart; a major earthquake occurs in sequence; and the series closes with after-shocks that appear almost incessant to the frightened people. Thus the habit is characterized by a succession of phases: (1) accumulation of strain which simulates inactivity; (2) phase of premonitory shocks of moderate intensity and small area; (3) a major relieving shock; (4) after shocks.

The California earthquake of 1906 was a relieving shock. It relieved the strain that had been accumulating for many years. It has been followed by the period of inactivity, which has lasted twenty years and may continue for some years yet, as the relief of strain was more complete than usual. Eventually there will follow a certain number of premonitory shocks. The heralds of another major earthquake. This forecast is the best we can make with our present knowledge of the actual phase, but it is limited to the area within which the 1906 earthquake was effective in relieving the strain. That area does not extend south of the latitude of Monterey. For southern California the last great relieving shock was in 1857 and the local disturbances of the last eight years, of which the Santa Barbara earthquake was the latest, are generally regarded by seismologists as premonitory incidents.

The earthquake habit is similar throughout all of California, but there are two provinces and the phases are not the same in the north and south.

As regards the area of disturbance in any given earthquake, much depends upon the phase to which it belongs. The premonitory shocks usually startle people throughout an area of one thousand to five thousand square miles. The relieving event may have a similar effect over twenty or thirty thousand square miles in California. The destructive intensity is confined to very much smaller areas in either case. In California, owing to the peculiar geological structure of the earth's superficial crust, the disturbance is confined to a long narrow strip in any particular district. The indications are that we have to expect a deep-seated, more far-reaching earthquake in southern California. It may not necessarily involve Los Angeles seriously, since that City is not near the great fault, the San Andres fault, but it is nevertheless likely to do much widespread damage in the south. Therefore, in response to questions which have come to me in my official relation with this problem, I have said to business men, bankers, insurance men and others connected with Southern California business, "I would be very conservative in my investments. I would be particularly conservative in the examination of buildings for investment or insurance, and, if I were building, I would see to it that I built right."

In the north, I think we have quite a long period to wait before we need expect a shock.

The Atlantic Coast presents an entirely different earthquake habit. The interval of accumulation appears to be exceedingly long, a century or more; the premonitory shocks, if they occur, are too weak to be strikingly evident; the relieving earthquake is not of excessive intensity at the surface, so far as we can judge on the available data; the after shocks are relatively few and weak. But the areas affected by the major shocks, such as that of Charleston, 1886, and of the St. Lawrence, 1925, are very large, amounting to several hundred thousand square miles. Hence we infer that the energy released at the deep-seated focus is great.

What is the earthquake habit of the Atlantic Coast? It is an obscure habit. We have no adequate records. Let me tell you a little story. A lady in the West wrote me last August, "Dear Sir: You predicted the Santa Barbara earthquake, you went down there to experience it. Perhaps you can tell me whether my daughters will be safe in San Francisco or not this Autumn." I said: "Dear Madam: I think your daughters will be safe in San Francisco this Autumn. We have studied earthquakes in California and we think we know something about them. I regret to say that we are not equally well informed regarding New England." (Laughter).

Now, that sounds like a joke. But it is a fact. In 1638 the old Pilgrim Fathers were shaken out of their boots. In 1755 Boston had a shock which, if it hit the brick buildings on the mud flats there, would do a great deal of damage. Those shocks will unquestionably be repeated, because the processes of nature go on through thousands and millions of years without any reference to whether we expect them or not. When Mr. Kelsey this morning said it was not considered worthwhile to look into the building codes of the East because it did not seem necessary, he made a statement which I cannot back up, because I know that your methods of buildings here are inadequate for the emergency which may come. Out of yonder window this morning I counted thirty-two water tanks which will go down through the fifteen or twenty-story buildings on which they stand at a very slight shock here in New York. I looked out of that window ten minutes ago and there are at least two chimneys that will fall some ten stories and pass through the roof of the lobby.

I do not expect a very severe shock near or in New York. Manhattan Island itself is a rock mass, which is extremely firm and in which even a severe earthquake would create only a shiver. Your great buildings would not be materially affected. But the tenement houses, the apartment houses, are sources of danger here, especially on soft ground, and such incidental structures as I have mentioned are a source of danger everywhere, even in the moderate shocks that are likely to come.

The student of these diverse earthquake habits is led to think that they are dependent on the magnitude of the masses involved and the rate of accumulation of energy in the mechanism. The masses moved in California are relatively small mountain-blocks, their resistance is not great, and the amount of energy which can gather behind them, so to speak, before they move is correspondingly moderate. But the snap which releases it is located not many miles below the surface and therefore causes rather violent, but local disturbances. From the frequency of the shocks it may be inferred that the source of energy is vigorous.

In the Atlantic region, on the contrary, the masses which are set in vibration are enormous; their resistance to displacement is proportionately great; the amount of energy required to overcome it is of similar magnitude; but the source would appear to develop it at a slower rate than beneath the Pacific. All these conditions suggest a relatively deep source of disturbance in a portion of the crust where the geologic forces are comparatively inactive. Various lines of geologic reasoning support this view, which, in turn, leads to the conclusion that the earthquake risk in the Atlantic border, though not negligible, is remote.

A study of the history of British earthquakes, which goes back a thousand years, gives somewhat similar assurance. The total number of sensible shocks of all degrees listed in 950 years is 1,191, a little more than one a year or about the same frequency as that experienced in New England during the last three hundred years. Yet in that period Great Britain has not been shaken severely, as countries about the Pacific Ocean have been repeatedly.

Davidson* lists twenty-two rather local shocks which shook down chimneys and cracked heavy masonry walls in England or Scotland during the period covered by reliable records, namely 974 to 1924 A.D. None of them, however, reached a high degree of violence or disturbed, with even such maximum intensity as they attained, any considerable area.

The contrast of the earthquake habit of the Pacific Coast with that of the Atlantic leads to the conclusion that in California insurance is a very important if not a necessary protection from the point of view of the insured, but should be given only with caution by the insurer; whereas on the Atlantic Coast the protection is desirable only under special circumstances and may be granted readily when wanted.

We may now turn to another phase of the insurance problem, which may be stated in the form of the question: What should be insured against earthquake hazard? Excluding accidents and lives as not pertinent to this occasion, we have primarily to deal with the earthquake risk to buildings.

Should all buildings be insured? By no means. There are those which are so located or so constructed that they need no earthquake insurance, as, for instance, a structure deeply founded on solid rock, or one isolated from the ground by some type of free foundation, or a light building that is so firmly braced and tied that it may be regarded as earthquake proof. One might almost as well insure against fire in the pyramids of Egypt.

On the other hand, there are buildings which are so poorly constructed that no insurance company should take the risk. A conspicuous type is the Class C structure designed as an apartment house or other tenant occupation, where the owner does not risk his own person and the object is to get the largest return on the least money. There are many such and they are those for which insurance is most urgently demanded. It should be denied or taxed out of existence.

It is evident that rational insurance demands discrimination among earthquake hazards as among fire hazards. Let us see if we can recognize some of the criteria that should be applied in making that discrimination.

I proceed to consider the factors that should enter into the rating of buildings to be insured.

First: On what material does a building stand? On a rock or a bowl of jelly, that is of mud? Evidently the vibrations to which it may be subjected will differ in the two cases, being harmless in the former, but decidedly dangerous in the latter because of their greater amplitude. There is a wide range of risk, from safety to probability of destruction, depending on the firmness of the foundation material whether it be rock, compacted soil, or soft mud. It is to be expressed by the "foundation coefficient," a figure that varies from 1 to 10.

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*Davidson, Charles, A History of British Earthquakes, Cambridge University Press, 1924.

Second: What is the form of the structure? Squareness and regularity make for stability. Disproportion and irregularity suggest danger. The principle that is here involved is that of the swing which the building may take as a whole or in its parts. Every structure has a natural period of swing, peculiar to itself, like a pendulum. If it be a unit, it swings all together and may stand. If it be composed of parts that sway with different periods, the several parts will wreck each other as they collide. It is thus with a house and its chimney. Or again, the natural period of swing of the structure may or may not be the same as that of the ground. If it is, then there is a tendency to exaggerate the swaying of the building as the repeated oscillations of the ground coincide with and increase its own movement. On the other hand, if the structure has a period which is longer or shorter than that of the ground then the repeated earthquake motions check the swaying of the building, which is less strained accordingly. This is one explanation of the fact that adjacent or nearby buildings of similar construction but of different forms may suffer very unequally in the same shock.

Third: Is the structure built with bearing walls or is it framed? Where the floor and roof loads are borne by the walls, insurance rates should be high. They may be lowered when the walls are firmly tied together at frequent intervals by continuous ties from outside to outside of the whole.

Fourth: Framed structures outside constitute by far the larger part of the values covered by earthquake insurance and are therefore the most important. As now constructed, they consist of flexible frames clothed by rigid, fragile walls. The designer proposes that the flexible frame shall stand. Whether it remains clothed with the rigid walls or not is relatively unimportant to him. It usually does not. His makeshift concession to custom and ignorance will not, in my judgment, survive the next great disaster to an American City. It is, however, an actual condition of construction from which damage will result and it is consequently an essential consideration in earthquake insurance.

Framed structures should be so designed that the building as a whole will be a unit. Both frames and walls should be safe up to some assumed stress. To accomplish this condition requires one or two things: Either the frame must be so rigid that it will take all of the stresses without distortion and thus protect the fragile walls against crushing effects; or the frame and walls must be one in the sense that they must be equally strong to resist distortion by the calculated stresses. This demands tensile strength in the panel walls, a condition now commonly lacking because brick and tile manufacturers continue to build as the Egyptians did.

That construction in which every part works is most nearly in accordance with the principles of architecture and is also, as a rule, the most economical in the end. It will ultimately prevail, but at present the weight of ancient practice and the lack of ingenuity in design stand in the way.

The materials of which a structure is built are usually given considerable weight in rating schedule, and not without reason; but they are not as important as good design and good construction. When properly designed a frame may be equally secure whether it be of wood, ferro-concrete, or steel; and if the design be good the walls also will be right.

For light houses, the wood frame is the best. It is cheap, can be adequately braced at an additional cost of less than one per cent, and can be made less susceptible to fire by the uses of metal lath and cement plaster. Ferro-concrete comes next for buildings of greater weight, but still within moderate limits of height. The tendency is to use it beyond safe limits for a material whose strength depends on the variable factors of honesty and workmanship. Rigid inspection is its only guarantee. The erection of a ferro-concrete frame for the upper stories on top of a steel frame should be absolutely condemned by insurance companies. The joint is apt to come at a point where the strain in a tall structure is a maximum and the difference in elasticity between the lower steel and upper concrete frames must introduce diverse movements that will wreck the building even in a slight shock. Steel is recognized as the aristocrat of structural materials. Unfortunately, like the aristocratic framework of society, it is prone to crush the plebian substance of the structure. In their own interest steel men should decline to furnish steel for the flexible type of frames and equally for their own standing with the public the manufacturers of brick and tile should refuse to allow the use of their products where they must inevitably be subjected to strains they cannot stand.

Intimately connected with earthquake insurance is the incidental fire risk. It might well be considered as a part of the coverage granted by the earthquake policy, since the fire in such case is a result of the earthquake. But, fortunately, there are lawyers who would in any particular case raise the question whether the earthquake had actually caused the fire and therefore whether the loss came within the scope of the policy. Hence it may be wise to regard fire as fire and to add an appropriate penalty to the premium in the fire policy for those districts where earthquake insurance is underwritten. Since the risk is incidental to the expected earthquake it shares the character of the shock in that it is a remote and occasional event and the penalty should accordingly be estimated as a fraction whose divisor is the number of years likely to elapse between shocks and whose numerator includes the area likely to be covered by the destructive activity. The penalty will commonly be found to be but a small fraction, in all probability, provided that all other factors, such as the type of building and the general conditions of the fire hazard have previously been taken into account.

I have reserved for the last, the most difficult problem of earthquake insurance, the problem of making it pay. It is too local, too uncertain, too concentrated in its losses to be good business, at least as it is now carried on. Jones, an old conservative, opulent underwriter, does not want it. He takes it under protest because he has to get competitive fire risks. He seeks to protect himself, limiting his liabilities, by insuring for short terms of a year or two, by betting as safely as he can. In fact he usually bets on a certainty, since he underwrites earthquake insurance only when the public demands it and that is after the event, when the hazard is negligible. Smith, a young, enterprising fellow, new in the business and in search of opportunities, thinks he sees one in this earthquake stuff. If he could only spread it, diversify it. He is wise enough not to imagine that he can build a strong line on public anxiety. That little earthquake up in Westchester the other day may serve Jones' turn, but Smith wants something that is founded on confidence, that will be lasting.

Looking over the field, Smith discovers that there are various accidents or incidents of nature that threaten lives and property; not only earthquakes but also tornadoes, hail-storms, lightning, floods, and so forth. He learns that each is a regional risk inasmuch as they rarely occur in the same areas, and they therefore as a whole constitute a widespread, diversified risk, such as that for which he is seeking. He recalls that similar risks are inclined under an "Act-of-God" policy in marine insurance. Why not write an "Act-of-God" policy on land? There are difficulties in the way.

Earthquake insurance is underwritten for very short terms only. The Acts-of-God are occasional and remote from the public thought. Nobody wants Smith's Act-of-God policy just for a year or two. And yet the Acts-of-God are as certain as death within a lifetime. He concludes to handle his new line as life insurance is handled on long terms and yearly, accumulating premiums. The business needs legislative authority. There is abundant precedent. He gets it. He is able to fix a low rate of premium on a twenty year term. He sends out his salesmen, well instructed in those things the old line companies do not know, natural phenomena and the history of their occurrence throughout the country. He builds a good business on the prosperity of the United States and he gives to that people a protection against acts of God that now annually desolate isolated communities for whom there is no help but charity, because big business lacks knowledge and imagination. It is for the young minds, the men of enterprise and vision, to study this problem on the basis of scientific data with a view to profit for themselves and service to the people.

Now, gentlemen, I want to say one word in behalf of the Society which I represent.

The Seismological Society of America is established for the purpose of research, to develop our knowledge of earthquakes, which God knows, is little enough in view of all that is to be learned; and also to do a certain work in protecting the people of the United States against their own ignorance, stupidity and carelessness.

In research we are trying to establish more instruments, seismometers, to give us local data. You ought to have them here in New York. You ought to have them in Boston. We are putting them in in San Francisco, with some help from the insurance companies, as well as from other business interests. We ask you for further support in that direction. Then, we are working for building codes, as your Committee stated this morning. We have to insert a clause in codes which will correct the trouble. It does not mention earthquake, but merely requires that there shall be adequate bracing and tying. We are working also against panic in schools and in buildings. My advice to everybody when an earthquake comes is to sit still and count sixty. You will then be safe.

In our program of research and education we need the support not only of the National Board of Underwriters, which we have had most cordially in matters of advice and suggestion, but for the individual backing that you gentlemen, who represent the brain and leadership of insurance, can give. (Applause).

I take pleasure in inviting you all to become members of the Society and thus to join in our work.

EXCERPTS FROM A TECHNICAL REPORT ON THE GOLDEN GATE BRIDGE

Prepared by Parsons, Brinckerhoff, Hall & MacDonald
Consulting Engineers - New York

In connection with consideration of feasibility of
installing rail rapid transit facilities on the bridge

October 3, 1955

Summary of Report

The Golden Gate Bridge still has a large reserve capacity. This is borne out by engineering studies which show that the vertical live loads will not reach the intensity provided for in the original design, and that higher unit stresses than were used in the design may safely be permitted in the materials of which the bridge is constructed.

Reserve Capacity of Bridge

The history of bridge design is one of continually improving analytical methods, of increasing the fund of research data, and of profiting by past experience. It is the intent of this section to examine the twenty-year old design of the Golden Gate Bridge in the light of present day knowledge, in order to determine if a reserve or unused capacity exists in the structure without permitting stress increases in excess of what is considered good practice today.

Standard Bridge Specifications for either highway or railway bridges are generally valid only for spans of less than 400 feet. For greater lengths, the designer must write his own specification based on his own experience, judgment, intuition, and ability to foresee future loading conditions. If the structure is not of record-breaking span or pioneering type, he has the precedent of others to guide him. However, the design of the Golden Gate Bridge, in one bold stroke, more than doubled the main span length of any bridge in existence except the George Washington Bridge, and it exceeds the span of the latter by 20 per cent. Accordingly, the designers had no precedent to help guide them; hence there was a need for conservatism in certain portions of the design such as the cables and stiffening trusses. The satisfactory performance of the bridge since it was constructed, together with the conservatism of the original design, form a basis for determining the reserve capacity of the structure.

To determine the reserve capacity, the design live loads and unit stresses will be re-evaluated in the light of experience and information that has accrued since the time of the original design. In the ensuing interval, truck sizes relative to axle and linear foot loads have become better defined, and studies have been made as to the actual values of highway loads that can be anticipated. Similarly, this experience period has produced evidence of satisfactory working stresses not available at the time of the original design. The paragraphs that follow will discuss these points.

Design Live Loads

It is now customary to use a design unit live loading on long span highway bridges below that given in the specifications of the American Association of State Highway Officials (A. A. S. H. O.). The standard design live load for an arterial highway such as is carried on the Golden Gate Bridge according to these specifications is the H20-S16 loading; for long loaded lengths H20 and H20-S16 are synonymous. However, as noted previously, standard specifications are limited to spans not longer than 300 feet to 400 feet. The paper, "Live Loading for Long Span Highway Bridges" (Trans. Am. Soc. C. E., Vol. 119, Page 981) shows that the average load per foot decreases as the span length increases and recommends decreasing the live load gradually until it is equal to seven-eights of the standard H20-S16 loading (without load concentration) for loaded lengths in excess of 1,200 feet. This procedure was followed in the design of the Richmond-San Rafael Bridge which is now nearing completion.

On the San Francisco-Oakland Bay Bridge a total load equivalent to seven lanes of H20 loading was spread over the nine highway lanes. While the conventional reduction factor for multiple loaded lanes is .75 in today's specifications, the San Francisco-Oakland Bay Bridge over-all factor was .715. Converting this 1931 loading to today's standards results in an equivalent single lane loading of H14.85 (without load concentration) in each lane. (The concentrated load portion of the A. A. S. H. O. Specifications accounts for only 5 per cent of the total moment on a 1,000-foot span and less than 3 per cent for 2,000 feet.) The Mackinac Straits Bridge (now under construction) has a central span of 3,800 feet. It will have cables and stiffening trusses designed for 2,000 pounds per lineal foot of bridge, equivalent to an H15.6 loading (without load concentration) in each of the bridge's four lanes.

The floor system, however, is designed for H20-S16 loading. The design live load for the suspension spans of the Chesapeake Bay Bridge, opened to traffic in 1952, is H15; the main span is 1,600 feet. The proposed live loading for long span highway bridges referred to in the previous paragraph, provides for about H17.0 loading on spans in excess of 1,200 feet.

The use of this slightly heavier load is predicated upon military convey loading. On bridges and especially on a monumental structure, such as the Golden Gate Bridge, the military would order increased vehicle spacing when so advised by the authorities responsible for the bridge.

As noted previously, the Golden Gate Bridge was designed for a moving load of 4,000 pounds per lineal foot of bridge, and of such length and position to produce maximum stress.

The above paragraphs establish the fact that the Golden Gate Bridge was designed for live loads considerably heavier than is customary now.

Combination of Dead and Live Load Stresses

It is generally accepted today that a bridge supporting a live load which is only a small percentage of the dead weight of the bridge may be designed with allowable unit stresses nearer the yield point of the steels used than would be proper if the live load were large relative to the dead load. This variation in allowable stress is justified by the fact that the dead load of a structure and the resulting stresses can be predicted quite accurately while the live loads can only be guessed at, guided by previous experience and the contemporary consensus of other engineers. Furthermore, the smaller the live load-dead load ratio, the smaller the possibility of any fatigue failure occurring.

The designers of the Golden Gate Bridge used very conservative stresses. Although the design specifications called for a heavy live load, they did not allow higher unit stresses than were then prevalent. Use of dual unit stresses would have been of little value in the suspension spans, since the ratio of the dead load to live load is very high, and since the stiffening truss was not designed for appreciable dead loads. Finally, the allowable unit stresses used are generally lower than A. A. S. H. O.'s present standards, even though the material used meets or exceeds today's specifications.

From the above, it can be seen that the Golden Gate Bridge specifications are indeed conservative.

Comparison of Unit Stresses with Those in a Similar Bridge

In evaluating a bridge, a comparison of the allowable stresses used in its design with those for similar bridges, especially any in the same area, serves as an effective method of ascertaining if an unsuspected reserve capacity exists in the first bridge. It should be cautioned, however, that loading and other conditions must be taken into consideration also. Following is a table comparing certain of the allowable working stresses for the Golden Gate Bridge and the San Francisco-Oakland Bay Bridge:

BASIC DESIGN UNIT STRESSES*

(In lbs. per sq. in.)

Stress	Golden Gate Bridge	San Francisco-Oakland Bay Bridge
Tension -		
Carbon Steel	16,000	22,000
Silicon Steel:		
Floorbeams & Stringers	22,000	28,000
Elsewhere	24,000	28,000
Cable Wire	82,000	82,000
Heat Treated Eyebars	27,000	34,000
Compression -		
Carbon Steel	16,000 - 70 l/r (14,000 maximum)	22,000 - 70 l/r (18,500 maximum)
Silicon Steel	24,000 - 100 l/r (18,000 maximum)	28,000 - 90 l/r (23,500 maximum)
Bearing -		
Carbon Steel	24,000	30,000
Silicon Steel	30,000	38,000
Power Driven Rivets	24,000	30,000

BASIC DESIGN UNIT STRESSES* (Cont'd)

(In lbs. per sq. in.)

Stress	Golden Gate Bridge	San Francisco-Oakland Bay Bridge
Shear -		
Carbon Steel:		
Girders	10,000	12,500
Pins	13,500	15,000
Silicon Steel	13,000	16,000
Power Driven Rivets	12,000	15,000

* For certain combinations of loading, these unit stresses are increased; e.g., the allowable unit stress for wind load plus dead load plus live loads is 133 per cent of the basic design unit stress.

The following table compares various minimum yield point strengths and minimum ultimate strengths for the steels in the Golden Gate and San Francisco-Oakland Bay Bridges.

SPECIFIED PHYSICAL PROPERTIES OF STRUCTURAL STEELS

	Minimum Yield Point Strength Lbs. per Sq. In.		Minimum Ultimate Strength Lbs. per Sq. In.	
	Golden Gate Bridge	SF-Oakland Bay Bridge	Golden Gate Bridge	SF-Oakland Bay Bridge
Carbon Steel	36,000	37,000	60,000	62,000
Silicon Steel	45,000	45,000	80,000	80,000
Cable Wire	160,000	145,000	220,000	215,000
Heat Treated Eyebars	50,000	50,000	80,000	80,000
Rivet Steel	30,000	30,000	52,000	52,000

A study of the two tables above shows that much more conservative stresses were used in the Golden Gate Bridge, than in the Bay Bridge. Although equal working stresses were used in the cables, the Bay Bridge used steel with a 10 per cent lower yield point. In effect, then, the Golden Gate cable stresses are 10 per cent lower than would be permitted by the stress-yield point ratio in the Bay Bridge cables. Similarly, the allowable unit stresses in the stiffening truss members are less than those in the Bay Bridge by 17 per cent in tension and 25 per cent in compression for the silicon steel and over 30 per cent less in tension or compression for the carbon steel.

Conclusion

The discussion above shows a considerable reserve capacity exists in the Golden Gate Bridge. This reserve results from the use of a live loading which investigation has proven to be excessive for today's traffic and from conservatism in the choice of unit stresses, as shown by the comparison of the design specifications of the Golden Gate Bridge with those for the San Francisco - Oakland Bay Bridge.

COMMENTS UPON DESIGN FOR EARTHQUAKES

From a Symposium on the Long Beach Earthquake of March 10, 1933
Heldin Los Angeles, California, on April 12, 1933, Contributed to by

Dr. H. O. Wood,
Head of Seismological Laboratory, Carnegie Institute

Dr. J. P. Buwalda,
Head of Dept. of Geology, Calif. Institute of Technology

Prof. R. R. Martel,
Head of Structural Engineering, Calif. Institute of Technology

Introduction

Shortly after the earthquake of March 10, 1933, a technical meeting was sponsored by the American Society of Mechanical Engineers at the Jonathan Club in Los Angeles, for the purpose of reviewing the cause, effect and scientific aspects of the quake. The primary purpose was to bring together a group of engineers and scientists for discussion of means by which construction design for earthquake resistance could be enforced in California.

Design for Earthquakes

It was the unanimous decision of the speakers, all of whom were renowned in their fields of scientific endeavor, that there need be no fear of earthquakes if structures are properly designed to withstand the forces of nature caused by shifting of the earth's surface, resulting from seismic action.

It was pointed out that most of the structural damage in the 1933 Long Beach earthquake was due to faulty design and construction. Major buildings constructed in Los Angeles since that time are of special earthquake-resisting design. The U. S. Post Office is a typical example, with its unusually heavy steel girders and sway bracing.

In commenting upon the subject, Dr. Buwalda said,

> "We must recognize that southern California will continue to have earthquakes. . . and we will have them sufficiently so that we should build against them.
>
> "If the Press would stress the fact that we will have earthquakes, the public would demand good buildings and would not leave it to the Code or Building Inspector to enforce good construction. It does not cost much more to build for earthquakes."

Professor Martel, in commenting upon the manner in which engineers could assist in the program of fostering earthquake resistant construction, said,

> "Perhaps the biggest contribution he can make is to help as a parent or a citizen to keep the interest in earthquake resistant construction from waning, as it has too consistently after all past earthquakes, with the hope that we will have such safe buildings that we may adopt as the slogan of Southern California: 'Come to California and enjoy the earthquakes'."

Professor Martel told a story of an American engineer visiting in Tokyo, who observed a steel-frame building under construction designed for a horizontal force 50 per cent greater than required by the Code. The building was to replace one that had been destroyed by the earthquake. The sizes of girders, beams and columns and their connections were much heavier than would be required in the United States for similar construction. Upon being questioned as to the necessity for such heavy design, the Japanese President of the company stated, "It would not be necessary now if we had done it before the 1923 earthquake."

Conclusion

Adequate design for earthquake resistance is not something new or recent. It was well known before the earthquake of 1923 in Tokyo and the 1933 earthquake in Long Beach. It was known, therefore, prior to construction of the Golden Gate Bridge.

In the same manner that more than ample provision was made for structural strength to withstand live, dead and wind loads, generous application was made of earthquake resistant design to ensure adequate stability of the bridge structure and its foundations under seismic stresses.

There are a number of steel framed buildings in San Francisco today that were adequately designed to withstand the earthquake of 1906, and continue to give servide today. By far, most of the damage of that quake was due to fire. In this connection, Dr. Buwalda said on April 12, 1933,

> "It is true that a large part of the destruction in San Francisco resulted from the fire, but it is equally true that if earthquake destruction is severe, it is usually followed by fire and there is no use trying to separate them. It happened in San Francisco, it happened in Tokyo. We should guard against both by earthquake resistant construction; it can be done and we should do it."

The Golden Gate Bridge is of massive steel and concrete. There is no fire hazard and the suspension design is sufficiently proportioned and inherently elastic to obviate the probability of serious physical damage under severe seismic acceleration.

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PART FIVE

EARTHQUAKE OF MARCH 22, 1957

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Golden Gate Bridge and Highway District

REPORT OF GENERAL MANAGER ON EARTHQUAKE OF MARCH 22, 1957

March 29, 1957

TO: Honorable Board of Directors

FROM: James Adam, General Manager

SUBJECT: Survey and Inspection of Bridge Properties following a series of Earthquakes, commencing Friday, March 22, 1957

Gentlemen:

Immediately following a sharp earthquake at 11:45 a.m. on Friday, March 22, 1957, a survey and detailed inspection was conducted of the bridge and District properties.

A similar general inspection has continued daily since that time because of numerous after-shocks, but there have been no indications of any damage.

The inspections have covered:

Item 1:

The inspections have covered:

(a) The four (4) main cable saddle castings located on the top of each tower leg of the San Francisco and Marin towers at elevation + 746.00.

(b) The base of both the San Francisco and Marin towers at elevation + 44.00.

(c) The San Francisco and Marin cable anchorages, including the eye bars and strand shoe assembly.

(d) The main bridge cable tie downs at Pylon S 1 and N 1.

(e) The 36 3/8" diameter main cable bands between San Francisco and Marin anchorages for a total distance of 15,300 feet on both cables.

NOTE: Repaired one (1) slightly bent hand-rope stanchion on the San Francisco back span west main cable at panel No. 5.

Item 2:

All buildings at the Toll Plaza, including toll booths, Round House Restaurant and powerhouse:

(a) About 20 windows were broken in the toll booths, and a few panes in the Administration building, as a result of the earthquake.

Item 3:

(a) A complete check of the expansion assemblies at the San Francisco and Marin towers: This included an inspection of the sliding blocks (attached to center span lateral system) and bearing plates for connection of center span laterals to the towers.

(b) Expansion floor gratings between Pylon S 1 and San Francisco side span and Pylon N 1 and Marin side span, and north and south expansion gratings for the arch span.

All expansion joints were found to be in satisfactory condition and not damaged whatsoever.

Item 4:

The two (2) service elevators, one in each of the two main towers of the Golden Gate Bridge:

(a) Car and counterweight guide rails and attachments, including safety mechanism, hoisting and compensating cables, compensating sheave assemblies and overhead sheaves. All were found to be satisfactory and not damaged.

Item 5:

The roadway sodium vapor lamps. Inspection revealed that no damage resulted from the earthquake.

Item 6:

A general inspection of the new Machine Shop and Garage Building, as well as the Administration Building, was made Monday, March 25, 1957, by J. B. Lundberg, Architect, and Mr. Edward M. Knapik, Civil Engineer, both associated with the office of Milton T. Pflueger, 580 Market Street, San Francisco.

Inspection included steel roof framing, bracing, column connections, footings, concrete walls and slab. No earthquake damage was found.

Item 7:

General inspection of the new travelling scaffolds on the under-structure of the suspender spans showed no displacement from the rails, and all rail clamps in place. There was no damage.

Item 8:

Bridge action caused by the earthquake:

(a) For bridge action see vibration recordings for Friday, March 22, 1957. The largest double vertical amplitude was recorded by the vibration instrument in House No. 1 on the west side of the San Francisco side span.

Record of Recording:

Maximum double amplitude 5 6/10 inches, frequency of vibration equals 20 per minute; period of vibration was 3.5 seconds, time 11:45 a.m.; wind direction southwest 10 m.p.h.

Item 9:

The survey and inspection of earthquake damage to District properties reveals that no damage, except minor glass breakage in toll booths and surface cracks in concrete, occurred.

Respectfully,

/s/ James Adam

James Adam
General Manager

REPORT OF CONSULTING ENGINEER ON EARTHQUAKE OF MARCH 22, 1957

Clifford E. Paine - Consulting Engineer
Fennville, Michigan

April 2, 1957

Mr. James Adam, General Manager
Golden Gate Bridge and Highway District
San Francisco, California

Dear Mr. Adam:

As requested, I have studied the vibration charts which you sent me. These are the charts which show vertical vibrations at instruments numbered 1, 3, 4, 5, 7, and 8 during the earthquake on the morning of March 22, 1957.

Instrument number 1 is at the center of the San Francisco Side Span. The double amplitude of record is 0.49 inch. The period is 3.5 seconds. The impressed vibration which would yield the above instrument recording is 5.6 inches. This is the vertical double amplitude obtained at the middle of the San Francisco Side Span.

Instrument number 5 is at the center of the Main Span. The double amplitude of record is 0.25 inch. The period of vibration is 3.33 seconds. The impressed vibration which would yield the above instrument recording is 2.9 inches. This is the vertical double amplitude at the middle of the Main Span.

The discussion is limited to instruments numbered 1 and 5 because charts from the other instruments contribute no pertinent, additional information.

Respectfully submitted,

/s/ Clifford E. Pain

Clifford E. Paine

REPORT OF ARCHITECT
ON EARTHQUAKE OF MARCH 22, 1957

Milton T. Pflueger, A.I.A., Architect
580 Market Street, San Francisco 4, California

March 28, 1958

Mr. James Adam, General Manager
Golden Gate Bridge & Highway District
Box 99, Presidio Station
San Francisco, California

Subject: Machine Shop & Garage Building
Golden Gate Bridge
INSPECTION FOR EARTHQUAKE DAMAGE

Gentlemen:

As requested, Mr. E. M. Knapik, Civil Engineer, and I inspected the Machine Shop & Garage Building for earthquake damage on Monday, March 25, 1957.

During inspection, cracks on the interior of concrete walls were brought to our attention. These cracks appear midway between columns and run vertically from windows to floor line.

In the original design of the building an endeavor was made to control cracks in exterior concrete walls by designing weakened plane joints midway between columns. These cracks appearing on the interior were anticipated and probably existed to a great extent prior to the earthquake and in no way affect the structural soundness or watertightness of the building.

No earthquake damage to the building was found.

Yours very truly,

/s/ J. B. Lundberg

REPORT OF U. S. COAST AND GEODETIC SURVEY ON EARTHQUAKE OF MARCH 22, 1957

Department of Commerce, U. S. Coast and Geodetic Survey
Seismological Field Survey, Old Mint Building
Fifth and Mission Streets, San Francisco 3, Calif.

28 March 1957

Re: EARTHQUAKE, SAN FRANCISCO BAY AREA, MARCH 22, 1957

In the Bay Area the Seismological Field Survey of the Coast and Geodetic Survey obtained strong-motion seismograph records of the main shock at 8 ground level stations, 7 upper floor stations, and 1 bridge station. Complete analyses of the seismograms will take several months. However, preliminary scaling of records indicate that at the nearest station to the epicenter (Golden Gate Park) there were several ground oscillations with maximum accelerations of about 4 ft/sec^2, associated with periods of about 0.15 sec. At ground level stations in downtown San Francisco records indicate maximum accelerations in the range 1 ft/sec^2 to 3 ft/sec^2 associated with a period of about 0.25 seconds.

/s/ William K. Cloud

WILLIAM K. CLOUD, Chief
Seismological Field Survey

STATE OF CALIFORNIA
GOODWIN J. KNIGHT, Governor
DEPARTMENT OF NATURAL RESOURCES
DeWITT NELSON, Director

DIVISION OF MINES
Ferry Building, San Francisco 11
OLAF P. JENKINS, Chief

MINERAL INFORMATION SERVICE

Vol. 10 May 1, 1957 No. 5

MINERAL INFORMATION SERVICE is designed to inform the public on the geology and mineral resources of California and on the usefulness of minerals and rocks, and to serve as a news release on mineral discoveries, mining operations, markets, statistics, and new publications. It is issued monthly by the California State Division of Mines. Subscription price, January through December, is $1.00.

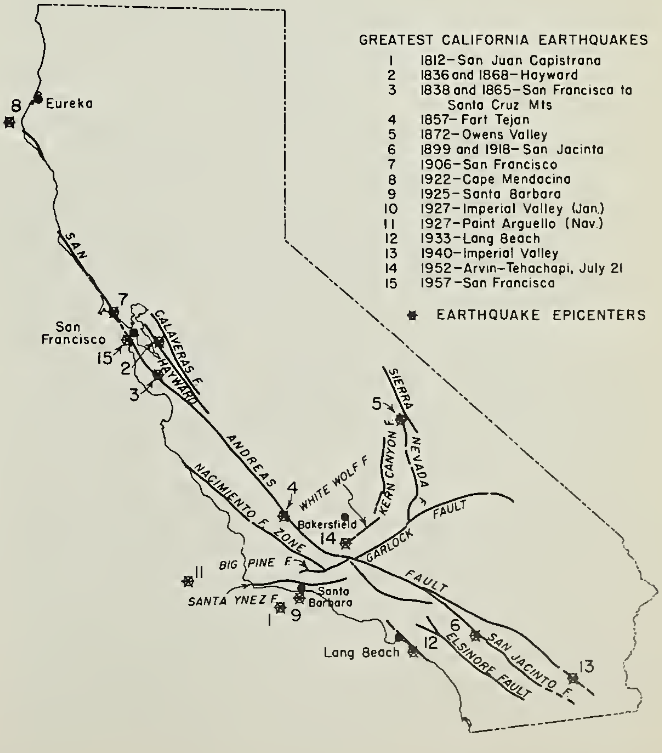
SAN FRANCISCO EARTHQUAKE OF MARCH 22, 1957

At about 11:45 on the morning of Friday, March 22, 1957, San Francisco and the Bay area experienced the strongest shock since the great earthquake of 1906. Actually, the energy involved and the damage resulting from the earthquake of 1906 were vastly greater, but the 1957 shock was sufficiently violent to make most people of the Bay cities acutely aware that a sharp earthquake had occurred. For a whole new generation of the local population it was the greatest earthquake they had experienced. No lives were lost, there were no serious injuries, and structural damage to homes was slight, although a large number of homes in the Westlake-Palisades-Daly City area suffered more or less superficial damage. On the modified Mercali intensity scale, which is a measure of the violence of shaking based on destruction produced by the ground waves, intensity VII (on a I to XII scale) was probably reached at some points in San Francisco. The Richter magnitude of 5.3 for this earthquake was computed from the seismograph records, and is a figure related to total energy of the earthquake liberated as seismic waves. Magnitude does not vary from point to point, but is a constant for a given earthquake. Magnitude of the 1906 earthquake, computed at 8.25, shows that the total energy involved in earth movements in the 1906 earthquake was enormously greater than the energy of the 1957 shock.

The most spectacular and interesting effects of the March 22 earthquake were visible along State Highway 1 between the southern city boundary of San Francisco and Mussel Rock, closing the road for about two weeks. The largest slide originated high on the steep slope east of the highway a few hours after the main shock and brought loosely consolidated sandstone and sand entirely across the road and down to the beach. Other damage on Highway 1 was extensive cracking along the seaward shoulder of the pavement, usually nearly parallel to the pavement edge. At Lake Merced, low, but steep, embankments around the southern margins of the lake crumpled into the lake, taking sections of road pavement and a paved path along with the unconsolidated soil and fill on which the pavement was laid. The greatest building damage was to homes in the Westlake Palisades tract of Daly City but damage was generally confined to exterior plaster. Building damage in the city of San Francisco was minor. Total damage of all types resulting from the earthquake probably does not exceed $1,000,000.

Causes of earthquakes

Earthquakes are vibrations transmitted as waves in the materials of the earth and so may originate from landslides, volcanic activity, explosions and any abrupt movements of masses of rocks. Nearly all destructive earthquakes have been the result of sudden movements of blocks of the earth's crust along breaks called "faults." Rock, which makes up the material of the earth, is elastic and may yield to stresses by slow creep over long periods of time. When the elastic limit of the rock is exceeded at any point, or friction along an old fault surface is overcome, an abrupt movement may take place causing an earthquake. The underlying reasons for accumulation of stresses in rocks of the outer part of the earth are little understood; however, such stresses and rock displacements are most frequent along the unstable margins of continental platforms and the ocean deeps. One such belt on the earth that is particularly active is the margin of the



Landslide near Mussel Rock, State Highway 1.

Pacific Ocean where many of the world's destructive earthquakes have been felt. California and Nevada, located in this seismically active Pacific belt, have had approximately 95 percent of the perceptible earthquakes in the United States.

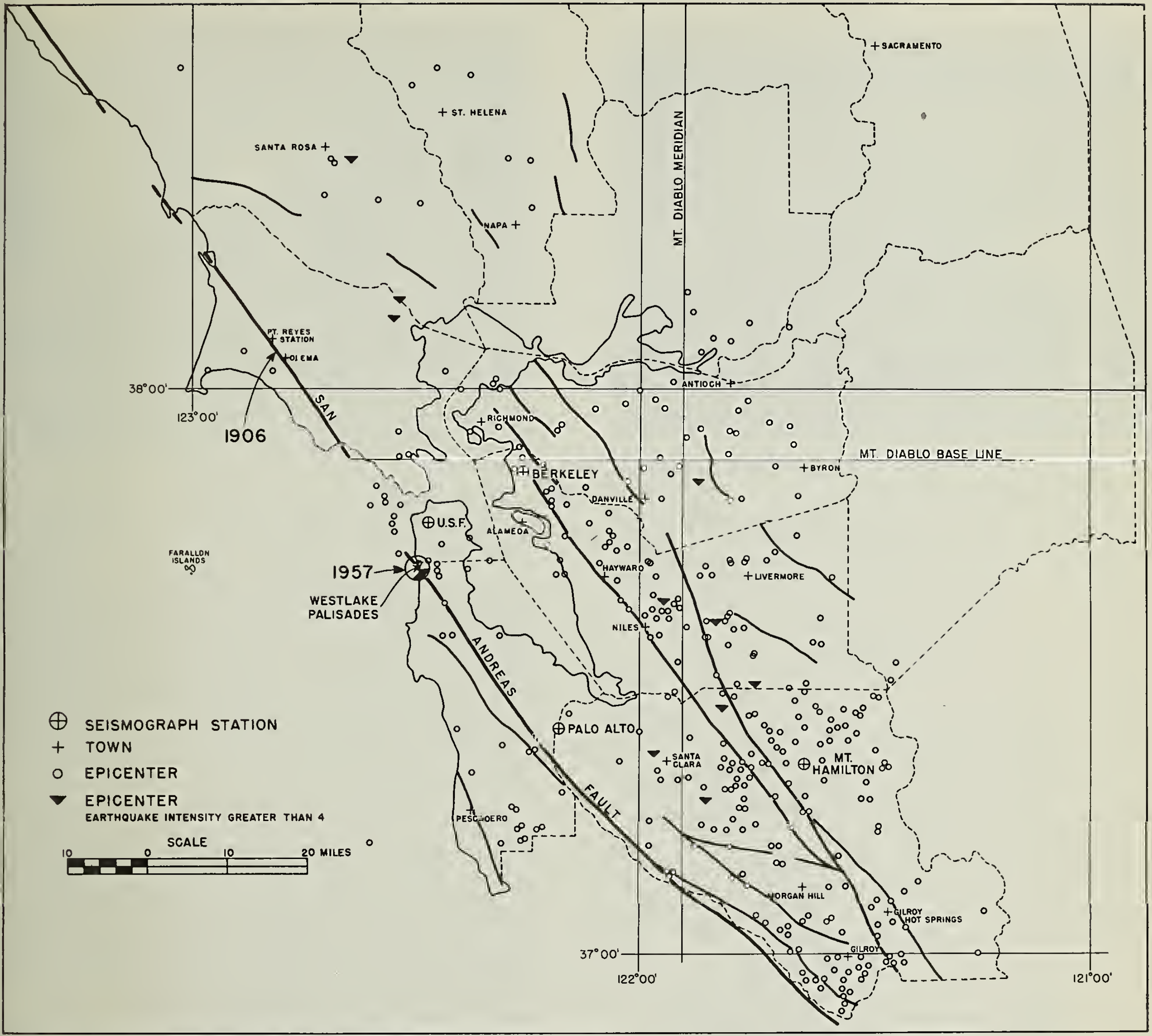
San Andreas fault

The San Andreas rift, or fault zone, is California's greatest active fault. It has been traced for more than 650 miles, from Point Arena to the Salton Sea. It is very clearly marked topographically by a series of narrow valleys, bays, small elongated lakes, sag ponds, and saddles, and geologically, by notable differences in rock types on opposite sides of the fault. From the ocean at Point Arena, the fault zone extends approximately S. 35° E. near the coast along Tomales Bay and enters the ocean again at Bolinas Bay. The fault does not traverse the city of San Francisco but is underwater, offshore, and appears on land again at Mussel Rock, just north of Sharp Park. It is marked by the Crystal Springs lakes on the San Francisco Peninsula, extends southeastward through San Juan Bautista, Parkfield, the Elkhorn Hills, Gorman on the Ridge Route, Palmdale, Cajon Pass, into the Imperial Valley east of the Salton Sea and probably into Mexico. There are two major bends in the San Andreas rift -- one at its junction with the Garlock fault in the Ridge Route area near Frazier Park and Gorman, and the other in a complex fault zone in the San Gorgonio Pass area-where the fault changes to a more nearly west-to-east trend.

Along the course of the San Andreas fault, repeated small displacements, resulting in earthquakes, have taken place at irregular, unpredictable, intervals over millions of years. The fault has thus become a zone of shattered and broken rock which may be more than a mile in width, often with rock formations of widely different type, structure, and age brought into contact. For example, the San Andreas fault along Tomales Bay separates granitic rock on the west from different altered sedimentary and volcanic rocks of the Francisco group to the east. Southeast of Mussel Rock, the soft, loosely consolidated brownish marine sedimentary rocks of the Merced formation (one to five million years old) are on the northeast; dark well-indurated rocks of the Franciscan group (about 100,000,000 years old) are on the southwest. The fault surface along which movement takes place at any particular time may approach a plane but is usually highly irregular and marked by the development of slickensides, breccia (broken rock), and gouge (clay-like, powdered rock). Fault surfaces in the San Andreas fault zone are near vertical and movements seem to be largely horizontal, in the right-lateral sense (east block moves southward in relation to west block). Surveys by the U. S. Coast and Geodetic Survey have shown that there is more or less continuous strain along this fault, with the east block creeping relatively southward at a rate of about 2 inches per year.

A number of great earthquakes, and many minor ones, have originated in the San Andreas fault zone and the related Hayward fault zone in the East Bay, in the San Francisco Bay area in historic times. In 1836 an earthquake in the Bay region opened ground fissures and was followed by strong aftershocks for a month. A severe earthquake damaged San Francisco Harbor, San Jose, and Monterey in 1838. Several earthquakes in 1864-65 brought widespread damage to the San Francisco-Santa Cruz area. One of California's major earthquakes originated on the Hayward fault in 1868; horizontal surface displacement was visible along this fault from San Leandro to Warm Springs. Thirty lives were lost in Hayward and San Leandro. The San Francisco earthquake of April 18, 1906 was one of the greatest shocks of California

Cracks in roadbed and shoulder, State Highway 1 near Mussel Rock.



Map of San Francisco Bay area showing principal active faults (heavy black lines) and the epicenters of earthquakes that occurred during the periods 1930-41 and 1947-48. The intensity of the various shocks is shown by the epicenter symbols listed in the legend to the left of the map. "1906" -- Center of greatest fault displacement (21 feet, horizontally) of San Francisco earthquake of 1906. "1957" -- Epicenter of San Francisco earthquake of 1957 within this circle of 3 mile radius. (Map modified from Division of Mines Bull. 154,p. 159, Fig.20)

history.Ruptures in the ground developed for a distance of 270 miles along the San Andreas rift from Fort Bragg to San Juan Bautista. Most of the displacement was horizontal(east side moved relatively southward) reaching a maximum of 21 feet, between Olema and the south end of Tomales Bay. Over 900 aftershocks were recorded during the following year. Between 500 and 1000 people in the Bay area lost their lives as a consequence of the earthquake and resulting fires, and property damage was estimated at one billion dollars. Computed Richter magnitude of the 1906 earthquake is about 8.25.The monumental 3-volume work of the State Earthquake Commission has given us a wonderful record of this earthquake, including a folio of maps showing the San Andreas fault the length of the state.

Seismograph data, University of California

The University of California, Berkeley, has for many years maintained several seismograph stations in the Bay area. Seismographs, which record earthquake waves, have been in use only about 70 years. The first seismographs in North America were set up in Berkeley and at Mt. Hamilton in 1887. The locations of the Bay area stations are shown on the accompanying map, brought up to date from the map published by the California Division of Mines in 1951 (Jenkins). It will probably be several months before the University will complete its seismic records and computations on the March 22 earthquake, but Dr. Don Tocher, of the University's Seismograph Station, has contributed data for the following paragraph, as of April 5th.

Road shoulder slump due to earthquake, Lake Merced, San Francisco.

A preliminary determination of the epicenter has been made from the arrival times of seismic waves at seismograph stations of the University at latitude 37° 42' north, longtitude 122° 31' west. This location is at sea near the southwest corner of San Francisco, and is consistent with the observed damage. Origin time of the shock was 11:44:21 P.S.T. The depth of the focus was probably less than 10 miles. The duration of the seismographic recording at Berkeley was at least 15 minutes; exact duration cannot be determined because of the numerous aftershocks which began soon after the main shock. The Seismological Laboratory of the California Institute of Technology at Pasadena has assigned a Richter magnitude of 5.3, based on the Pasadena seismograms alone. All seismographs in the San Francisco Bay region went off scale because of their proximity to the shock, hence their records cannot be used for magnitude determination. The following table lists all shocks of this series with magnitudes of 3.5 or above which have occurred to date.

Origin Time PST	Richter Magnitude
March 22	
10:48 (10:48 AM)	3.8
11:44 (11:44 AM)	5.3
13:07 (1:07 PM)	3.5
13:18 (1:18;PM)	3.5
15:14 (3:14 PM)	4.3
16:27 (4:27 PM)	3.9
March 23	
00:13 (12:33 AM)	4.2
04:54 (4:54 AM)	3.5

Strong motion-records, U.S. Coast & Geodetic Survey

The coast and Geodetic Survey's program of earthquake investigation consists in collecting descriptive and statistical information on earthquakes, in measuring destructive earthquake motions with special seismographs, and in analyzing the data for information of scientific and engineering value. The Survey maintains a network of strong-motion seismograph stations that operate only when strong seismic motion automatically triggers the instruments. Several such strong-motion stations are located in the Bay area. The information obtained is used to give comprehensive pictures of the intensity distribution (usually in terms of the Modified Mercali Intensity Scale). William K. Cloud, Chief, Seismological field Survey, U. S. Coast and Geodetic Survey, contributed data for the following paragraph, based on information to March 28.

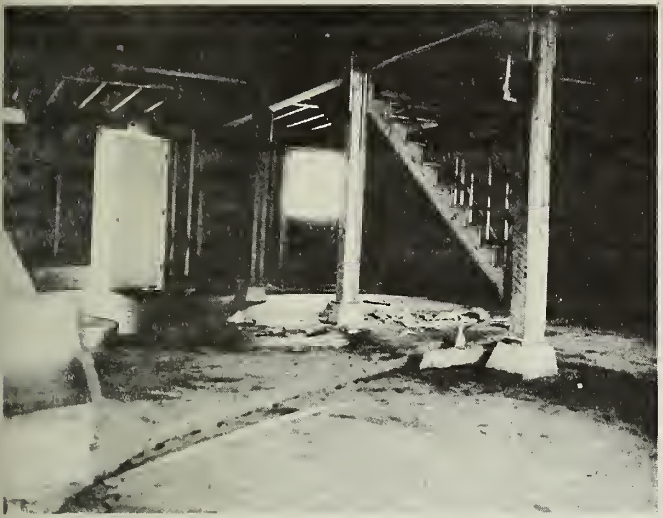
In the Bay Area the Seismological Field Survey of the Coast and Geodetic Surveyobtained strong-motion seismograph records of the main shock at eight ground level stations, seven upper floor stations, and one bridge station. Complete analyses of the seismograms will take several months. However, preliminary scaling of records indicates that at the nearest station to the epicenter (Golden Gate Park) there were several ground oscillations with maximum accelerations of about 4 ft/sec^2, associated with periods of about 0.15 sec. At ground level stations in downtown San Francisco records indicate maximum accelerations in the range 1 ft/sec^2 to 3 ft/sec^2 associated with a period of about 0.25 seconds.

Structural damage

The earthquake of March 22 was not particularly intense and overall damage was small. Within the past three years there have been eight earthquakes in the California-Nevada area as strong or stronger (California: Watsonville, 1954; Eureka, 1954; San Jose, 1955; Walnut Creek, 1951; Imperial Valley-Baja California, 1956. Nevada: Fallon, two in 1954; Dixie Valley-Fairview Peak, two major shocks on December 16, 1954). The Pacific Fire Rating Bureau, San Francisco, has supplied the following data from a survey conducted by their earthquake engineers, Karl V. Steinbrugge, Vincent Bush, and Edwin Zacher.

Building damage was strongest in the Westlake Palisades tract of Daly City which is located on a terrace several hundred feet in elevation along the Pacific Ocean. The tract comprises about 450 wood-frame dwellings, one and two stories high; all recently built. An estimated 50 percent of the houses in this tract were damaged, with the loss per dwelling estimated to average less than $200. Damage was generally confined to the exterior plaster of the first-story front wall (see photo).

Earthquake damage to exterior wall and steps by earthquake. Westlake-Palisades district, Daly City.



Crack in concrete floor of house caused earthquake. Westlake Palisades district, Daly City.

Damage to dwellings in San Francisco was noted throughout the city but was probably most extensive in the western portions. An accurate statement cannot be made until the results of the 2,000 questionnaires returned to the U. S. Coast and Geodetic Survey are processed. Plaster damage and some brick chimney damage were common. In general, the damage was slight and unsightly, represented by the minor cracks and spalls in plaster and paint, but in the thousands of buildings affected represents a large sum.

Quite a number of multistory reinforced concrete apartment buildings are located in the Lake Merced area. No significant structural damage has been found in the four Stonestown apartment buildings; these buildings are ten stories high. The multistory Park Merced Towers, which are structurally similar to the Stonestown buildings, had no reported structural damage. The Park Merced Towers had less glass damage than the Stonestown structures. These buildings were designed to resist earthquake forces, and the rigid type of construction which was used undoubtedly reduced partition damage to the observed negligible amounts. Multistory buildings in downtown San Francisco, some 6 miles away from the previously mentioned buildings, swayed considerably. However, the earthquake forces were generally not strong enough to cause any but slight plaster damage.

San Francisco airport is located on former mud flats. The ground in the vicinity of the airport buildings and the airline service buildings has been settling for years. Underground piping damage in the form of leaks and breaks was found in at least six locations, and more may be uncovered. Earthquake damage to underground pipelines is to be expected in the former marshland and tideland areas around San Francisco Bay. One airplane hanger building had damage to several concrete walls, and damage to the roof bracing system. The structure has been somewhat weakened, but collapse is not feared. Reasons for this damage have not been determined, but ground settlements are suspected among other factors.

A 1,600,000 gallon reinforced concrete water reservoir, partly underground and located on the highest point of the Westlake Palisades tract, cracked along one side and apparently along the bottom. Soil conditions are suspected, but a conclusive explanation cannot be given until repairs are started.

Some mention should be made of the C-Class (C-Class structures are defined as those having wood floors and roof, with exterior bearing walls of Masonry of any type) structures which form a class usually sustaining the heaviest damage in a major earthquake. Defective workmanship was noted in one instance where damage occurred. Some stone fell from a gable in Colma. In general, the earthquake was not strong enough to cause damage to this class of construction.

Daly City had 3 pipe breaks in the Westlake Palisades tract, and none elsewhere. Pipe damage occurred to the sewage treatment plant at Linda Mar, and again settlements were the major factor. The National Board of Fire Underwriters reported that San Francisco's high pressure system and cisterns were not damaged. Pipeline damage has not been compiled for other areas, but no indication has been received that much damage has occurred elsewhere.

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REPORT OF PACIFIC FIRE RATING BUREAU

465 California Street - San Francisco 4, California

April 12, 1957

TO MEMBERS AND CALIFORNIA SUBSCRIBERS

Preliminary Report

SAN FRANCISCO EARTHQUAKE OF MARCH 22, 1957

SUMMARY

Over-all damage in this earthquake was small and follows from the fact that the earthquake was of no more than moderate intensity. Within the past 3 years there have been 8 earthquakes in California and Nevada which have been as strong or stronger.

Dwelling damage generally did not exceed one or two per cent of insured value in the heavier shaken regions, except for several isolated cases. Underground pipelines were damaged in a few instances, and damage could be laid to the poor soils or shifting soils.

Non-earthquake resistive buildings with masonry bearing walls, which are vulnerable in a strong earthquake, were generally undamaged.

Multi-story buildings near the epicentral region sustained virtually negligible damage due to their earthquake bracing. Several instances of multistory building damage were reported in downtown San Francisco approximately 8 miles from the epicentral region.

Individual losses to buildings were usually minor, but the sum of many thousands of these minor losses will place the over-all earthquake loss at some figure estimated to be under $1,000,000. Conventional earthquake insurance policies have a minimum deductible of 5%, and insurance losses will probably not be particularly significant.

TABLE 1

E. Q. Location	E. Q. Date	Maximum Modified Mercalli Intensity	Richter Magnitude	Remarks
Watsonville, Calif	April 25, 1954	VIII	5.3	Unpublished report on file.
Fallon, Nevada	July 6, 1954	VIII-IX	6.8	Published report in "BSSA," 46:1/40
Fallon, Nevada	Aug. 23, 1954	VIII-IX	6.8	Published report in "BSSA," 46:1/40
Dixie-Fairview, Nevada:				
1st shock	Dec. 16, 1954	VIII-X	7.2	These earthquakes were a few minutes apart and in the same general locality. Report to be published in "BSSA."
2nd shock	Dec. 16, 1954	VIII-X	7.1	
Eureka, California	Dec. 21, 1954	VII	6.5	Published report available shortly.
San Jose, California	Sept. 4, 1955	VII	5.5	Unpublished report on file.
Walnut Creek, California	Oct. 23, 1955	VII	5.4	
Imperial Valley & Baja, California	Feb. 9, 1956	Max. not established	6.8	Geological and seismological report in progress.
San Francisco, Calif.	Mar. 22, 1957	VII (tentative)	5.3	

Notes:

1. "BSSA" refers to "Bulletin of the Seismological Society of America."
2. For a brief summary of major earthquakes in the San Francisco Bay Area, see "History of Earthquakes in the San Francisco Bay Area." Available from Earthquake Department, PFRB.

INTRODUCTION

The considerable interest in this earthquake has been indicated by the newspaper reaction in the form of headlines. However, the earthquake was neither particularly intense nor damaging. Table 1 compares the intensities and magnitudes of recent California and Nevada earthquakes. It will be noted that eight earthquakes have occurred within the last three years which are as strong or stronger than the latest one.

Table 1 clearly indicates that the press gave excessive emphasis to the recent earthquake. In addition, our preliminary survey of building damage has revealed no major losses.

Information from the University of California and California Institute of Technology would tentatively place the instrumental epicenter of the earthquake on that portion of the San Andreas Fault located in the ocean just off Daly City or just off San Francisco's Lake Merced district. The observed damage pattern would indicate a similar location for the field epicenter.[1]

The San Andreas fault enters the Pacific Ocean an estimated mile or more south of the nearest damaged dwelling area. No surface faulting has been reported, and none should be expected from an earthquake this small. It might also be added that proximity to a fault is not necessarily the most important factor in earthquake damage.[2]

It appears that the earthquake was strongest in that part of Daly City fronting on the Pacific Ocean. This area is known as the Westlake Palisades tract. As the accompanying map indicates, this region is just southwest of the San Francisco city limits.

The Westlake Palisades tract at present contains about 450 wood frame dwellings, about 90% of which are two stories high. All have been built within the last few years. An estimated 50% of the dwellings in this tract were damaged. Damage was generally confined to the exterior plaster of the first story front wall as indicated in the accompanying illustrations. Since there was usually no plaster or other finish in the interior of the first story of the two story dwellings, loss per dwelling has been estimated at $200 maximum, except where structural damage was involved. There are several reasons for this damage pattern. Earthquake forces are stronger in the first story. Secondly, the lack of partitions in the first story plus large wall openings in the front of this story caused the observed damage. Structural damage occurred to several dwellings in this tract. Soil conditions throughout this region are sandy, and slides in the steeper regions have occurred in the past. It is interesting to note that virtually identical dwellings located in the adjoining tracts to the east suffered materially less plaster damage.

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[1] "Field epicenter" is determined by observed damage; "instrumental epicenter" is determined by seismographic instruments. These two epicenters do not necessarily coincide.

[2] See "Earthquake Faults and Underwirting Faults" by Steinbrugge and Bush. Copies available from the Earthquake Department, PFRB.

Minor plaster damage to dwellings was common throughout Daly City. Forty-four damaged brick chimneys were pulled down by the fire department between March 22nd and March 26th, with undoubtedly many more damaged or destroyed. Brick chimneys in the area are few, due to the use of patent flues. At this date it is not known if damage was sustained by the terra cotta within the patent flues.

San Francisco - Dwellings

Plaster damage, and some brick chimney damage, was noted throughout San Francisco. It may have been more extensive throughout the western portions of the city. An accurate statement cannot be made until the results of the two thousand questionnaires returned to the U. S. Coast and Geodetic Survey are processed.

In general, the damage was slight but unsightly, and was represented by minor cracks and spalls in plaster and paint. While individual losses in this regard are negligible, this loss multiplied by many thousands of affected dwellings could represent a major insurance loss if earthquake insurance were carried on each dwelling and also carried without a deductible. It is premature to speculate on the actual number of dwellings having evidences of plaster cracking and similar minor damage.

San Francisco - Multi-story Buildings

A number of multi-story reinforced concrete apartment buildings are located in the Lake Merced area. No significant structural damage has been found in the four Stonestown apartment buildings which are ten stories high. Glass damage was considerable and formed an interesting damage patter; over-all partition damage was negligible. The multi-story Park Merced Towers, which are structurally similar to the Stonestown buildings, had no reported structural damage. The Park Merced Towers had less glass damage than the Stonestown structures.

The foregoing buildings were designed to resist earthquake forces, and the rigid type of construction which was used undoubtedly reduced partition damage to the observed negligible amounts.

Multi-story buildings in downtown San Francisco, some 6 miles away from the previously mentioned buildings, swayed considerably. However, the earthquake forces were generally not strong enough to cause any but slight plaster damage. One interesting exception was a 10 story steel frame office building on piling; damage to brick and tile non-structural partition walls was most pronounced in the first story and in the stairwell walls in the middle stories. Also, a 7 story hotel had damage in all stories at the locations where the original building adjoined its annex. This latter structure sustained similar damage in the October 23, 1955, Walnut Creek earthquake. Several other instances of plaster and partition damage were noted in other buildings, but over-all damage is not large. Numerous structures had no observable cracks.

San Francisco Airport

San Francisco's airport is located on former mud flats. The ground in the vicinity of the airport buildings and the airline service buildings has been settling for years. Underground piping damage in the form of leaks and breaks was found in at least 6 locations, and more may be uncovered. Earthquake damage to underground pipelines is to be expected in the former marshland and tideland areas around San Francisco Bay.

One airplane hanger building had damage to several concrete walls, and damage to the roof bracing system. The structure has been somewhat weakened, but collapse is not feared. Reasons for this damage have not been determined, but ground settlement is suspected among other factors.

Other Structures

A 1,600,000 gallon reinforced concrete water reservoir, partly underground and located on the highest point of the Westlake Palisades tract, cracked along one side and apparently along the bottom. Soil conditions are again suspected, but a conclusive explanation cannot be given until repairs are started.

Some mention should be made of the C-Class[3] structures which usually sustain the heaviest damage in a major earthquake. Defective workmanship was noted in one instance where damage occurred. Some stone fell from a gable in Colma. In general, the earthquake was not strong enough to cause damage to this class of construction.

Public schools, constructed under the Field Act,[4] had no structural damage. Minor non-structural damage was reported in several cases. The earthquake was not severe enough to damage the non-Field Act schools, and is consistent with the lack of damage to commercial buildings.

Underground Utilities

Daly City had 3 reported pipe breaks in the previously mentioned Westlake Palisades tract, and none elsewhere. Damage at the San Francisco Airport has already been mentioned. Pipe damage occurred to the sewage treatment plant at Linda Mar, and again settlement was the major factor. The National Board of Fire Underwriters reported that San Francisco's high pressure system and cisterns were not damaged.

Pipeline damage has not yet been compiled for other areas, but we have received no indication that much damage has occurred elsewhere.

Sprinkler Systems and Roof Tanks

A separate report is being prepared by the PFRB Sprinklered Risk Department

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[3] C-Class structures are defined as thos having wood floors and roof, with exterior bearing walls of masonry of any type.

[4] "Field Act" is a California law requiring strong earthquake bracing in public schools built since 1933.

With reference to roof tanks, damage at 3 of the 4 reported locations was due to rust penetrating the tank shell, defective piping, or causes other than strong earthquake stresses in the tank.

Interesting damage did occur to one elevated roof tank. While the tank had been strengthened to resist strong earthquakes some 25 years ago, the lack of maintenance has permitted corrosion of the bracing rods and other connecting members to the point that the tank was virtually a collapse hazard prior to the earthquake. There is evidence that some critically located bolts were lacking or rusted into two parts. It is remarkable that damage was merely confined to leakage through the tank shell and to the adjoining balcony girder.

FINAL REPORT

The California Division of Mines is planning to publish a detailed report on the earthquake, and it is expected to contain sections on geology, seismology, and structural damage. The structural portions will be prepared by the Earthquake Department of the PFRB, and this publication will constitute our final report.

The publication date is many months away, and suitable notice will be given at a later time.

ACKNOWLEDGMENTS

The following persons and organizations generously provided their assistance and facilities: Dr. Don Tocher of the University of California, the National Board of Fire Underwriters, the Board of Fire Underwriters of the Pacific, the General Adjustment Bureau, Mr. William Cloud of the U. S. Coast and Geodetic Survey, and the California Division of Architecture. The Daly City authorities, especially Fire Chief Elmer Kennedy, have been most helpful with their assistance. Dr. Gordon Oakeshott of the California Division of Mines provided information on the location of the San Andreas Fault.

Mr. Vincent Bush and Mr. Edwin Zacher of the Bureau's Earthquake Department materially assisted in gathering damage data and in writing this report.

Karl V. Steinbrugge
Chief Earthquake Engineer

PART SIX

QUALIFICATIONS OF ENGINEERS AND SCIENTISTS

1. Joseph B. Strauss, Engineer
2. O. H. Ammann, Engineer
3. Charles Derleth, Jr., Engineer and Professor
4. Leon S. Moisseiff, Engineer
5. Andrew C. Lawson, Geologist and Professor
6. Clifford E. Paine, Engineer
7. Perry Byerly, Seismologist and Professor
8. Charles M. Gilbert, Geologist and Professor
9. Francis J. Turner, Geologist and Professor
10. Rear Admiral William M. Gibson, Engineer

PART SIX

ATTAINMENT RECORD OF CONSULTING ENGINEERS AND SCIENTISTS PARTICIPATING IN DESIGN, CONSTRUCTION AND CONSULTATION SERVICES OF

OF THE GOLDEN GATE BRIDGE

1. JOSEPH B. STRAUSS, Bridge Engineer, Chief Engineer and Chairman of Engineering Board for design and construction of Golden Gate Bridge.

   Degree of C.E., University of Cincinnati, 1892; instructor at University of Cincinnati, 1893; Doctor of Science, University of Cincinnati, 1930; president and chief engineer, Strauss Engineering Corporation, bridge engineers, established 1894; inventor and designer of Strauss bascule and lift bridges.

   In charge of design and construction of approximately four hundred bridges all over the world. Co-designer Montreal South Shore Bridge across St. Lawrence river for Canadian Government.

   In charge of design and construction of the following major structures: Columbia River Longview bridge, Longview, Washington; Illinois River bridge, Peoria, Illinois; Mississippi River bridge, Quincy, Illinois; Independence-Liberty Bridge, Independence, Missouri; Outer Drive bridge, Chicago, Illinois.

   Consulting Engineer to Republic of Panama on Panama highway bridges, and to Governments of Santo Domingo, Norway, Denmark, Egyptian State Railways, etc., etc.

   Consulting engineer to Arlington Memorial Bridge Commission on Arlington Memorial bridge, Washington, D. C.

   Consulting engineer to Port of New York Authority on Hudson River bridge at 179th Street, New York City, and Bayonne Arch, Bayonne, New Jersey.

   Member: American Society of Mechanical Engineers, Western Society of Engineers, American Railway Engineering Association, American Society of Military Engineers, American Society for Testing Materials, Corporation of Professional Engineers of Quebec, Canada, Engineering Institute of Canada, Royal Society of Arts, Chicago Association of Commerce.

2. O. H. AMMANN, Bridge Engineer, Consultant and Member of Engineering Board for design and construction of Golden Gate Bridge.

   Degree of C.E., Swiss Federal Polytechnic Institute, Zurich, 1902. Connected with investigation and design of Queensboro bridge, N.Y., Quebec bridge, St. Johns River bridge, and C. & O. River bridge, Sciotoville. Assistant Chief Engineer, Hellgate bridge, New York. Chief engineer of bridges, Port of New York Authority.

   In charge of Outer Bridge crossing, Perth Amboy, Goethals bridge at Elizabeth, Bayonne Arch across Kill van Kull, 179th Street bridge, New York (3500-ft. suspension).

Now chief engineer, Port of New York Authority, in charge of plans, construction and maintenance and all Port Authority projects including vehicular tunnel under Hudson river at Weehauken (Holland tunnel).

Member: American Society Civil Engineers.

3. CHARLES DERLETH, JR., Bridge Engineer, Consultant and Member of Engineering Board for design and construction of Golden Gate Bridge.

   Degree of C.E., Columbia University, New York, 1896.

   Engineer of design for City of New York on: Manhattan bridge, Queensboro bridge, Williamsburgh bridge strengthening.

   Consulting designing engineer on: Delaware River bridge, Ambassador bridge at Detroit, Maumee River bridge at Toledo, Port of New York Authority, Hudson River and Kill van Kull bridges, Chicago World's Fair 1933.

   Member: Joint Committee on Concrete and Reinforced Concrete, American Society of Civil Engineers, American Railway Engineers Association, American Society for Testing Materials.

4. LEON S. MOISSEIFF, Bridge Engineer, Consultant and Member of Engineering Board for design and construction of Golden Gate Bridge.

   Degree of C.E., Columbia University, New York, 1895.

   Engineer of design for City of New York on: Manhattan bridge, Queensboro bridge, Williamsburgh bridge strengthening.

   Consulting designing engineer on: Delaware River bridge, Ambassador bridge at Detroit, Maumee River bridge at Toledo, Port of New York Authority, Hudson River and Kill van Kull bridges, Chicago World's Fair 1933.

   Member: Joint Committee on Concrete and Reinforced Concrete, American Society of Civil Engineers, American Railway Engineers Association, American Society for Testing Materials.

5. ANDREW C. LAWSON, Consulting Geologist.

   Degree of A. B. University of Toronto, 1883, A. M. 1885. John Hopkins University, Ph. D., 1888. University of Toronto, D. Sc. (Hon.) 1923.

   Geologist, Geological Survey of Canada 1882-90.

   Professor of Mineralogy and Geology since 1890 and dean of the College of Mining, 1914-1918, University of California.

   Delegate to the GeologicalCongress, London, 1888, St. Petersburg, 1897, Toronto 1913, Madrid, 1926.

   Chairman of the California Earthquake Investigation Committee, 1906.

   Fellow, Geological Society of America, A. A. A. S. President, Seismological Society of America, 1909-10.

Member, American Institute of Mining and Metallurgical Engineers, fellow, American Academy Arts and Sciences; member, National Academy of Science, Society of Economic Geologists.

Chairman, division of geology and geography, National Research Council, 1923.

Published numerous geological papers and monographs.

6. CLIFFORD E. PAINE, Bridge Engineer

Engineer of the Golden Gate Bridge and Highway District.

Degree of C. E., University of Michigan, Ann Arbor, Michigan, 1911.

Structural designer with American Bridge Company, Ambridge, Pa., and design engineer with Strauss Bascule Bridge Company, Chicago, 1911-1927 (except for military service, 1918-1919).

Vice President and Managing Engineer, Strauss Engineering Corp., Chicago, 1927-1935; Vice President, Strauss & Paine, Inc., Chicago, 1935-1938.

In direct charge of design and construction of movable and long-span bridges and principal Assistant Engineer on Golden Gate Bridge during construction.

Consulting Engineer in private practice, 1938-1953. Chairman of Board of Engineers investigating the Tacoma Narrows, Wash., Bridge collapse in 1940 for insurance companies. Consulting Engineer for U. S. Navy at Pearl Harbor, Honolulu, throughout World War II.

Consulting Engineer and Chairman of Board of Engineers investigating proposed improvements to the Golden Gate Bridge 1951-1953; appointed Engineer, Golden Gate Bridge and Highway District, 1953.

Member, American Society of Civil Engineers and Western Society of Engineers. Registered Engineer in States of Illinois, Michigan, Texas, Wisconsin, Iowa and California.

7. PERRY BYERLY, Seismologist, Professor of Seismology and Director of Seismographic Station at University of California, Berkeley.

Seismographic station, University of California, Berkeley (4), Calif. Seismology. Clarinda, Iowa, May 28, '97; married '41; children, 3. A. B., California, 1921; Fellow 1920-1924; A. M., 1922; Ph D. (Seismol. 1924); instructor Physics, Nevada, 1924-1925; in charge seismographic station, California, 1925-1950; Director 1950 on. Assistant Prof. Seismology 1927-1931, Assoc. Prof. 1931-1941, Professor 1941 on. Guggenheim Memorial Foundation fellow. Cambridge 1928-1929, and 1952-1953, with Office Sci. Research & Development, 1944, U. S. A., 1917-1919 National Academy; Seismological Society, (Secretary 1930-); Fellow Geological Society; Soc. Explor. Geophys. Union. California Seismology; Interests: earth structure; roots of mountains, energy in earthquake waves. Office: Bacon Hall, University of California, Berkeley (4), California.

8 CHARLES M. GILBERT, Geologist and Assoc. Professor of Geology at University of California, Berkeley.

2519 Etna Street, Berkeley, Cal. Physical Geology. Washington, D.C., May 22, 1910. A.B., Cornell, 1933; Phd., California, 1938. From Instructor to Assoc. Prof. in Geology, Cal., 1938 - ; Geological Society; Society of Econ. Paleontology and Mineralogy; Asn. Petrol. Geol.; Areal Geology; sedimentary petrography.

9 FRANCIS J. TURNER, Geologist and Professor of Geology at University of California, Berkeley.

Dept. of Geological Sciences, University of California, Berkeley (4), Cal. Petrology. Auckland, New Zealand, April 10, 1904; married 1930; 1 child; M.Sc.; New Zealand 1926; D.Sc. (Petrol.) 1934; Yale, 1938-1939. Lecturer, Geologist, Otago, New Zealand 1926-1934; sr. Lecturer, 1936-1946; Assoc. Prof., California, 1946-1958, Prof., 1948 --. Sterling fellow, Yale, 1938-1939. Fellow, Geological Society; Fellow, Miner. Soc; Geophys. Union; Fellow Geol. Society of London; Fellow of Royal Society of New Zealand. Interests: Petrology of ultra basic rock in New Zealand; metamorphic petrology in general; fabric of deformed rock; fabric of artificially deformed rocks and minerals; 3-axis universal stage for petrographic microscope - igneous petroligy in general.

10. WILLIAM M. GIBSON, Hydrographic and Geodetic Engineer, Retired Rear Admiral U.S. Coast and Geodetic Survey.

Consultant in Geodesy.

Bachelor of Science in Civil Engineering, Colorado State University, Fort Collins, Colorado, 1924.

Served continuously for seven years from September 1924, in Coast and Geodetic Survey, along Atlantic Coast and Philippine Islands; six years cooperative state work in eastern and middle Atlantic states.

1936-1940 -- organization and charge of computing office for City of New York, New York, N.Y.

1940-1950 -- surveying and charting in Pacific Ocean areas, including combat conditions with Pacific Fleet, under direction of Office of the Chief of Naval Operations.

1950-1953 -- Operations Officer of basic science activities of national and international scope for U.S. Coast and Geodetic Survey in Washington, D.C.

1954-1958 -- charge of U.S. Coast and Geodetic Survey, San Francisco

Professional affiliations: American Society of Civil Engineers
Society of American Military Engineers
Geological Society of America
Seismological Society of America
Naval Institute

Source of information

Note (a) (1) to (5) incl., from Report of the Chief Engineer, Vol. I, August 27, 1930.

(b) (6) from Board of Registration for Civil and Professional Engineers, State of California.

(c) (7) to (9), incl., from American Men of Science, 1955 Edition.

(d) (10) furnished upon request of District.

PART SEVEN

MAPS AND DRAWINGS

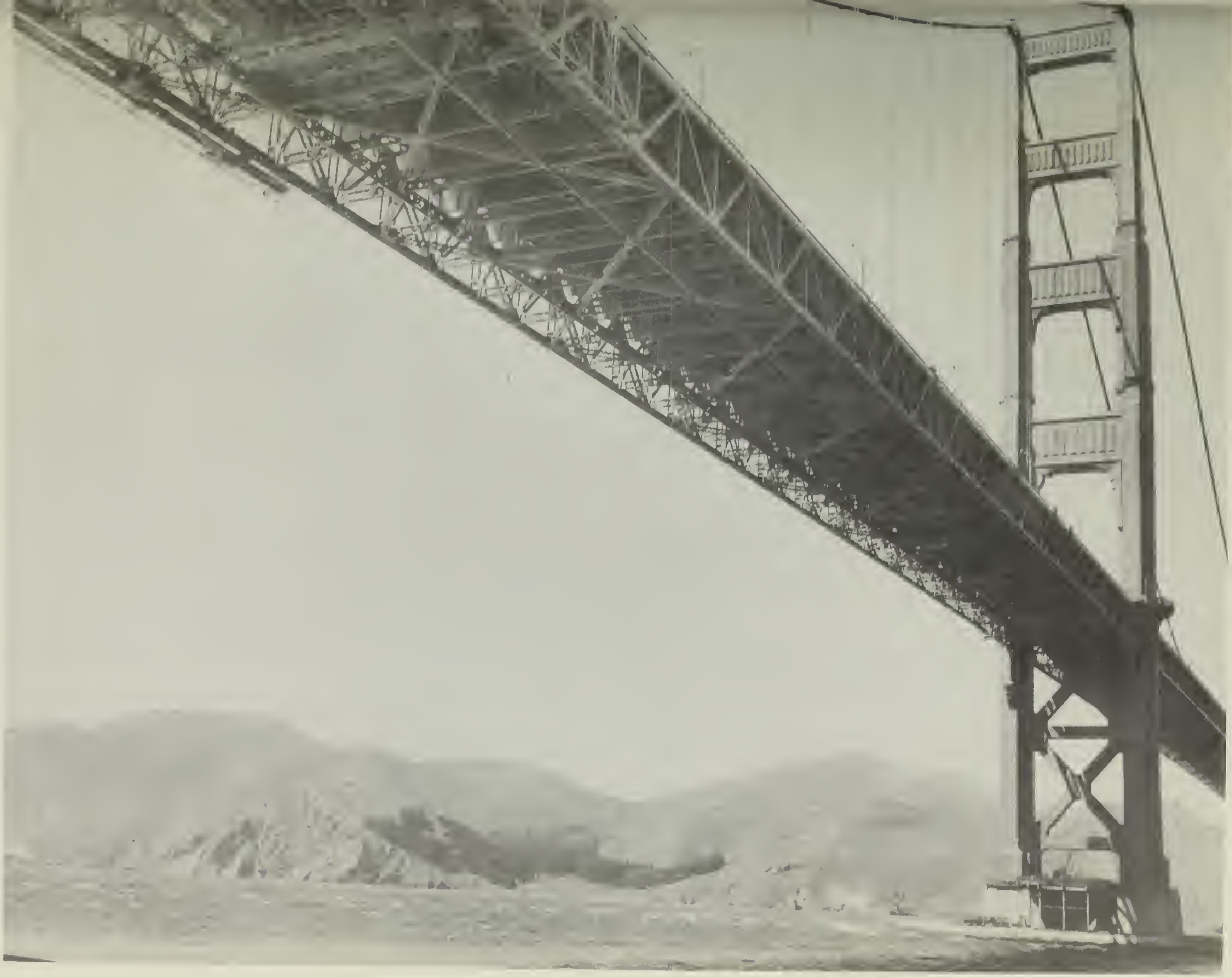
Report Reference:

		Page
A.	Photographs of Bottom Lateral Bracing Part 3, Section E, Page 47	126
B.	Submarine Maps Part 4, Section E, Page 74	127
C.	Report of Pacific Fire Underwriters Part 5, Section F, Page 113	128
D.	Structural Design Part 2, Various Sections	129

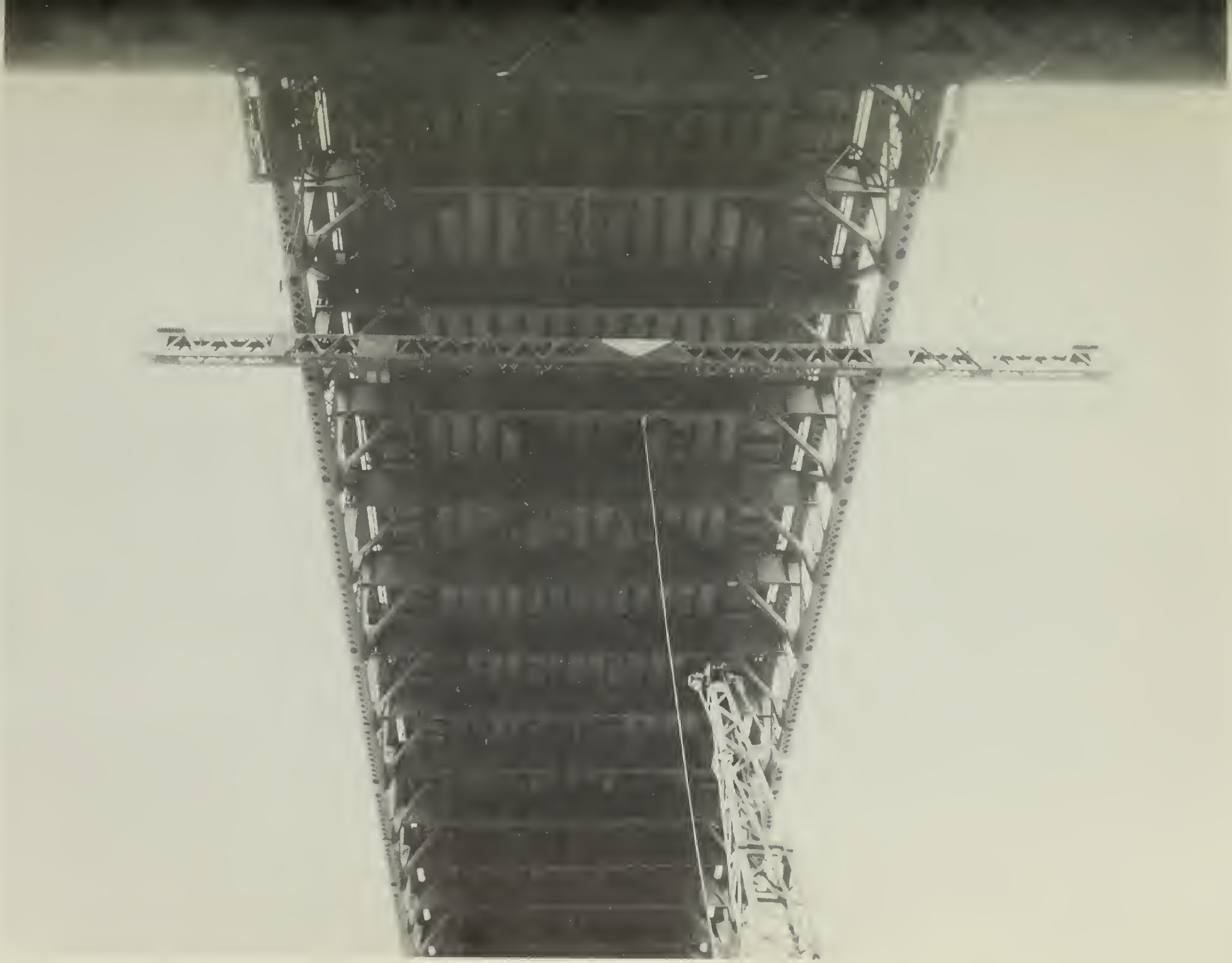
PART 7, SECTION A

PHOTOGRAPHS OF BOTTOM LATERAL BRACING, BEFORE AND AFTER INSTALLATION

(Refers to Part 3, Section E, Page 47)



Underside of Bridge Deck Structure After Installation of Bottom Lateral Bracing



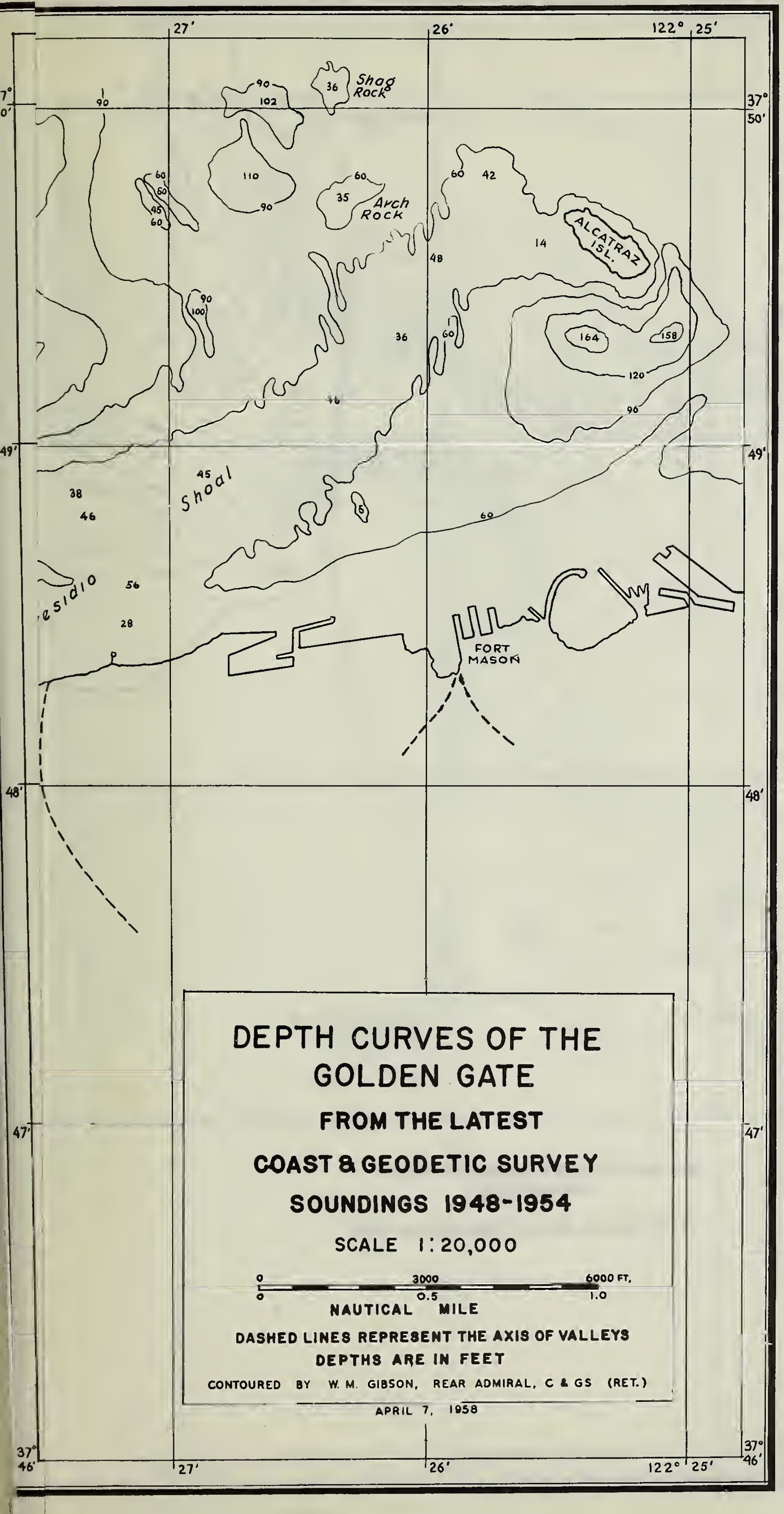
Underside of Bridge Deck Structure Prior to Installation of Bottom Lateral Bracing

PART 7, SECTION B

MAPS OF SUBMARINE TOPOGRAPHY IN THE VICINITY OF THE GOLDEN GATE BRIDGE

(Refers to Part 4, Section E, Page 74)

FIG. 1



DEPTH CURVES OF THE GOLDEN GATE

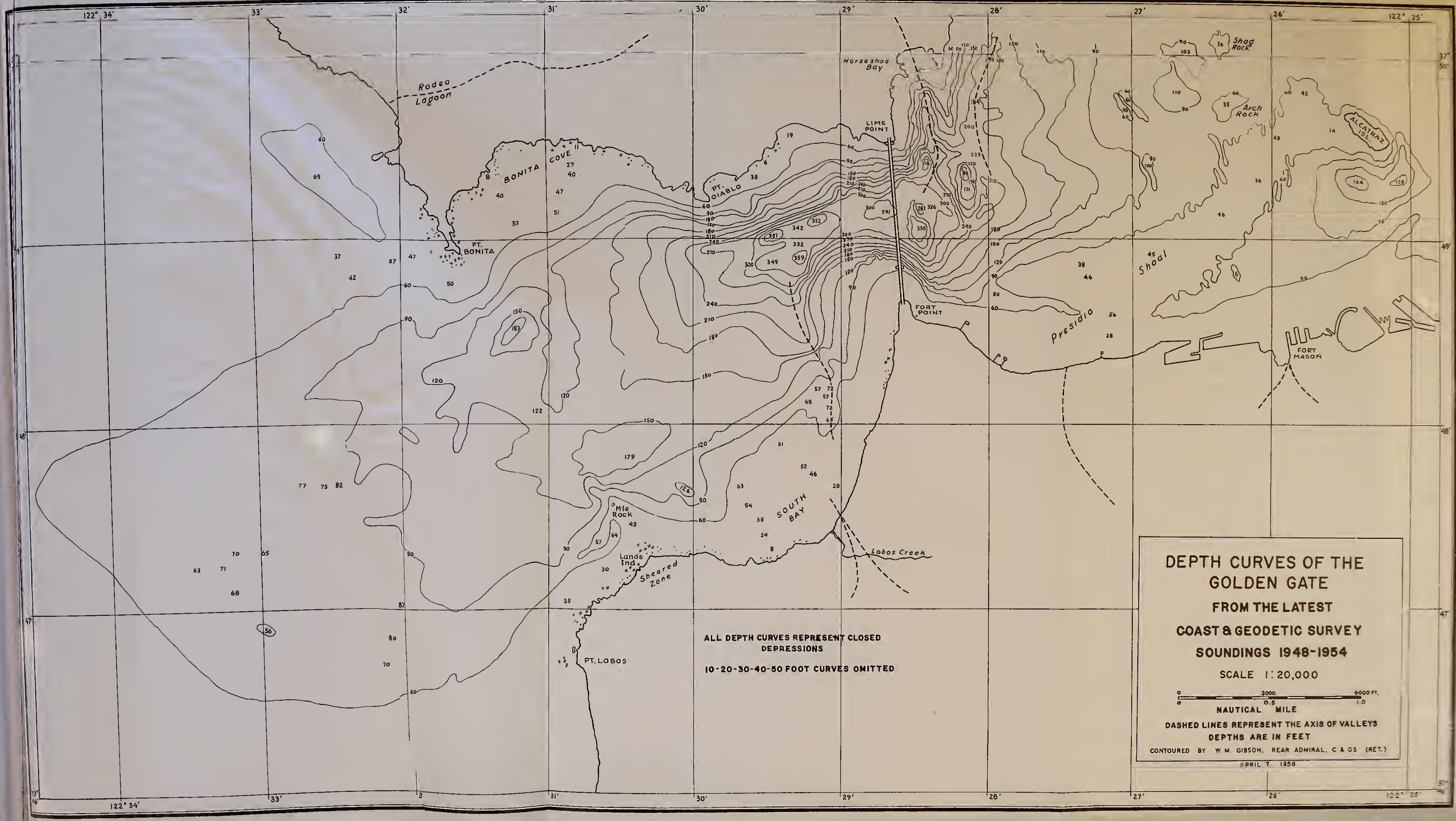
FROM THE LATEST COAST & GEODETIC SURVEY SOUNDINGS 1948-1954

SCALE 1:20,000

0 3000 6000 FT.
0 0.5 1.0
NAUTICAL MILE

DASHED LINES REPRESENT THE AXIS OF VALLEYS
DEPTHS ARE IN FEET
CONTOURED BY W. M. GIBSON, REAR ADMIRAL, C & GS (RET.)
APRIL 7, 1958

FIG. 1



DEPTH CURVES OF THE GOLDEN GATE
FROM THE LATEST
COAST & GEODETIC SURVEY
SOUNDINGS 1948-1954

SCALE 1:20,000

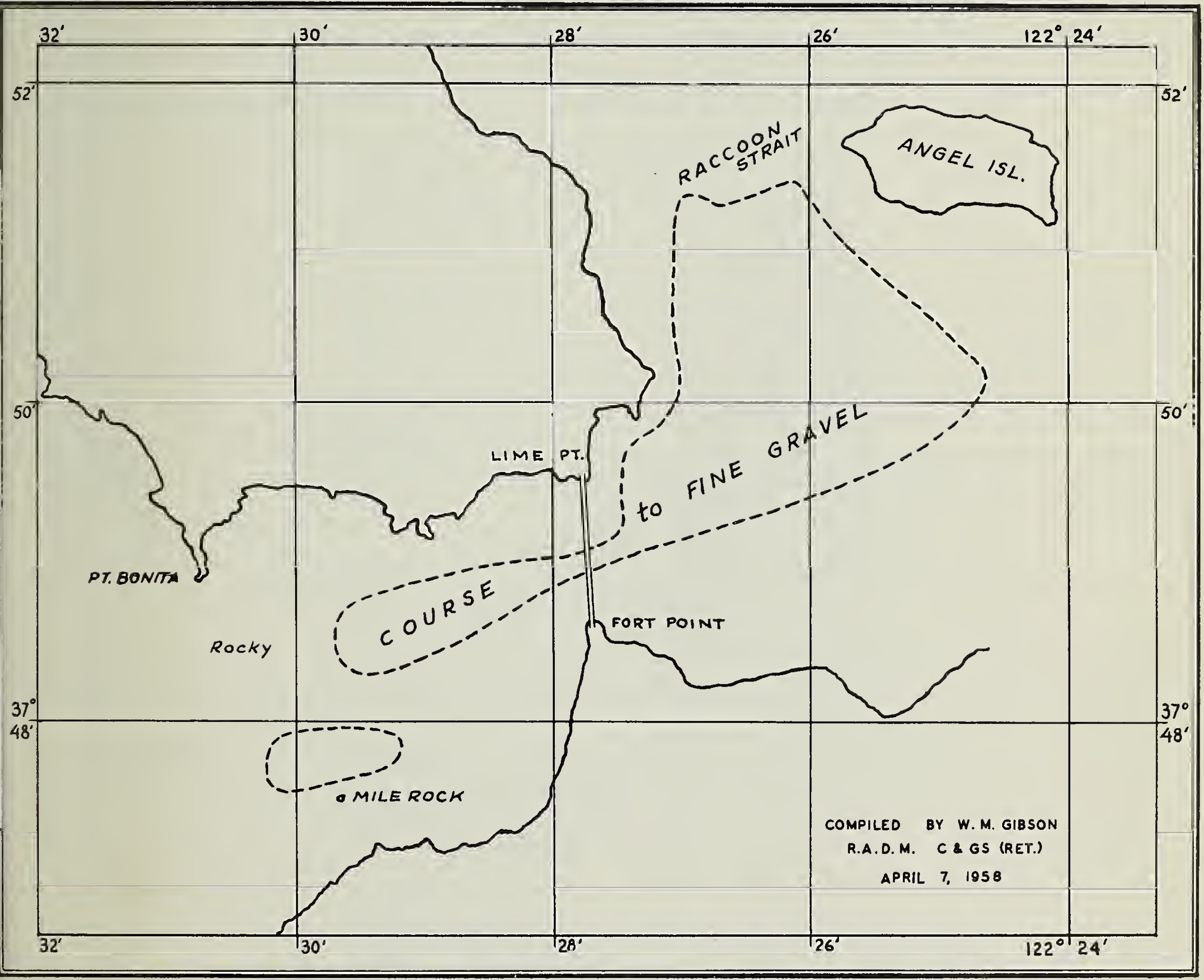
DASHED LINES REPRESENT THE AXIS OF VALLEYS
DEPTHS ARE IN FEET

CONTOURED BY W. M. GIBSON, REAR ADMIRAL, C & GS (RET.)
APRIL 7 1958

ALL DEPTH CURVES REPRESENT CLOSED DEPRESSIONS

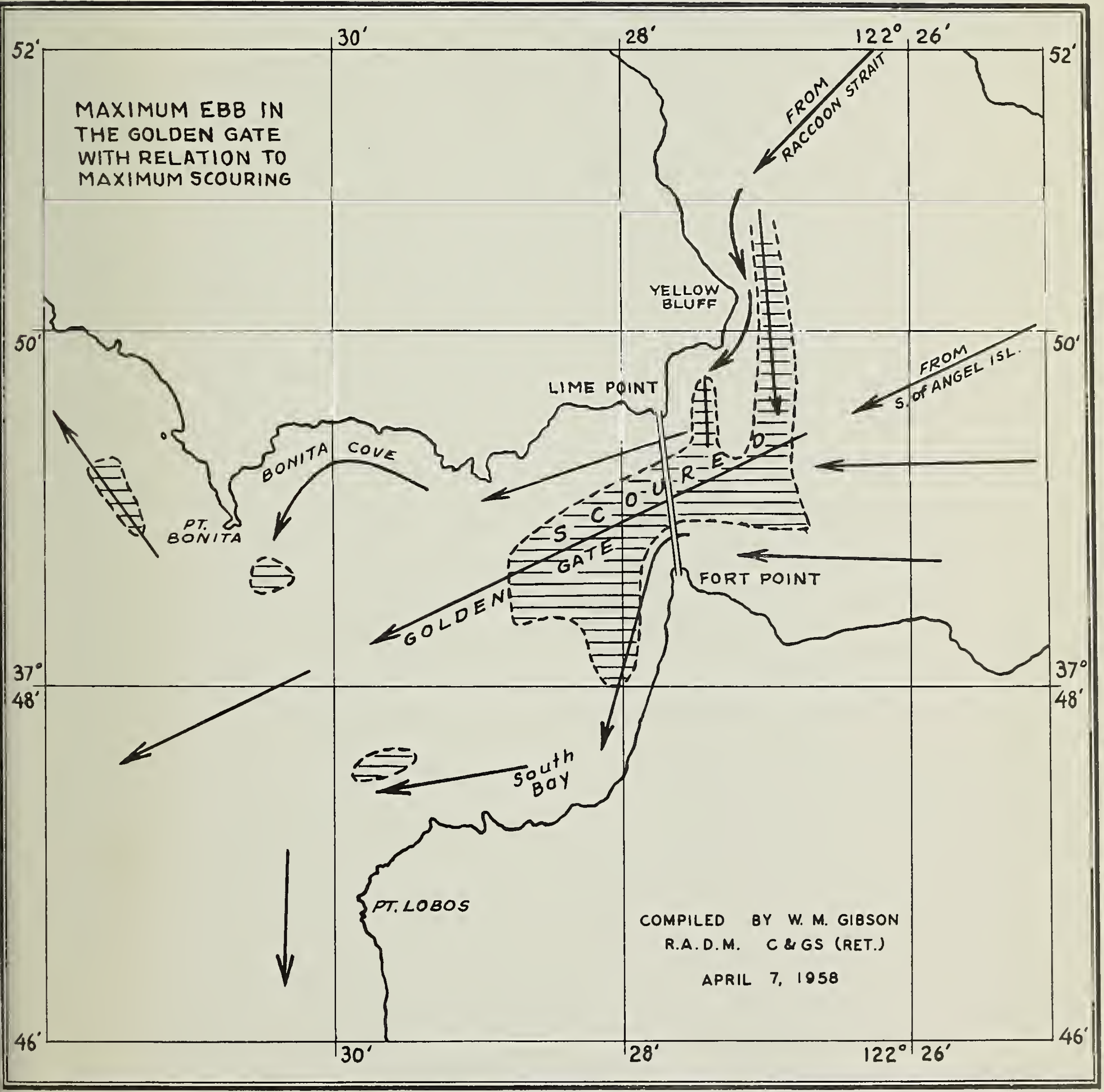
10-20-30-40-50 FOOT CURVES OMITTED

FIG. 2



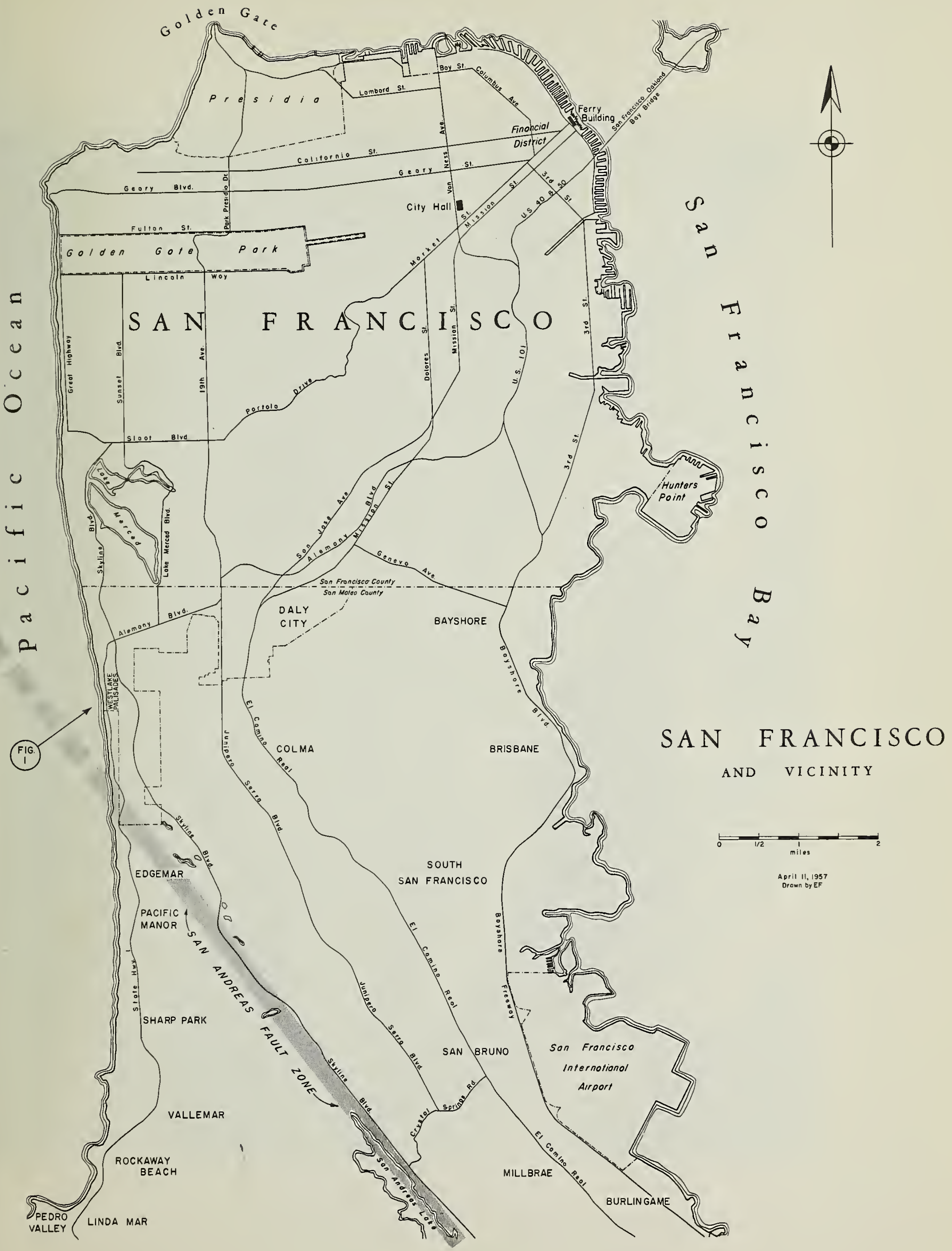
COMPILED BY W. M. GIBSON
R.A.D.M. C & GS (RET.)
APRIL 7, 1958

FIG. 3


MAXIMUM EBB IN THE GOLDEN GATE WITH RELATION TO MAXIMUM SCOURING
52′
30′
28′
122°
26′
50′
37°
48′
46′
FROM RACCOON STRAIT
YELLOW BLUFF
FROM S. of ANGEL ISL.
LIME POINT
BONITA COVE
PT. BONITA
S C O U R E D
GOLDEN GATE
FORT POINT
South Bay
PT. LOBOS
COMPILED BY W. M. GIBSON
R.A.D.M. C & GS (RET.)
APRIL 7, 1958

MAP AND PHOTOGRAPH OF SAN FRANCISCO SHOWING LOCATION OF SAN ANDREAS EARTHQUAKE FAULT

(Refers to Part 5, Section F, Page 113)



SAN FRANCISCO

AND VICINITY

April 11, 1957
Drawn by EF

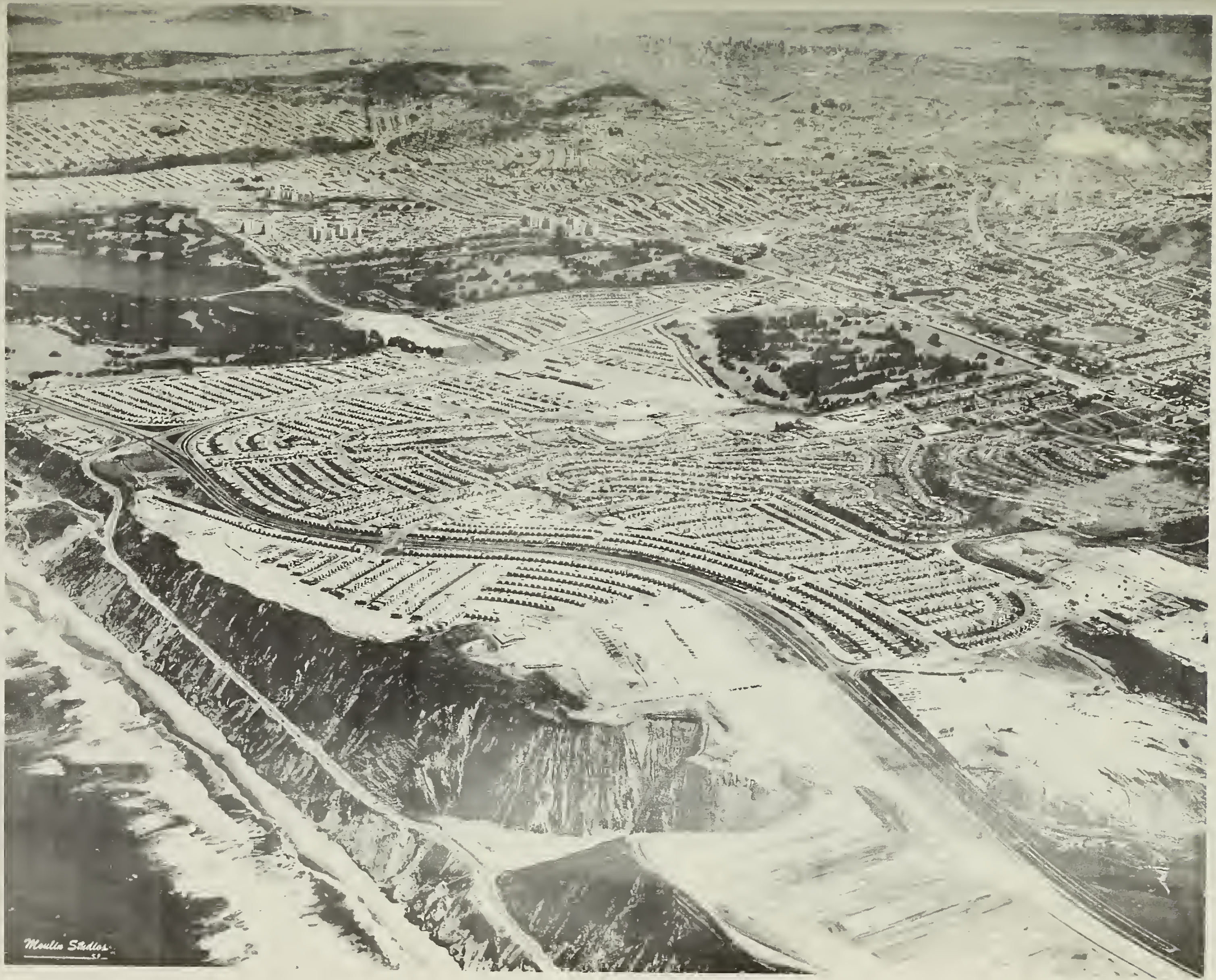


FIGURE 1

Daly City in foreground, with San Francisco's financial district at top center. Looking northeast. Pacific Ocean at lower left.

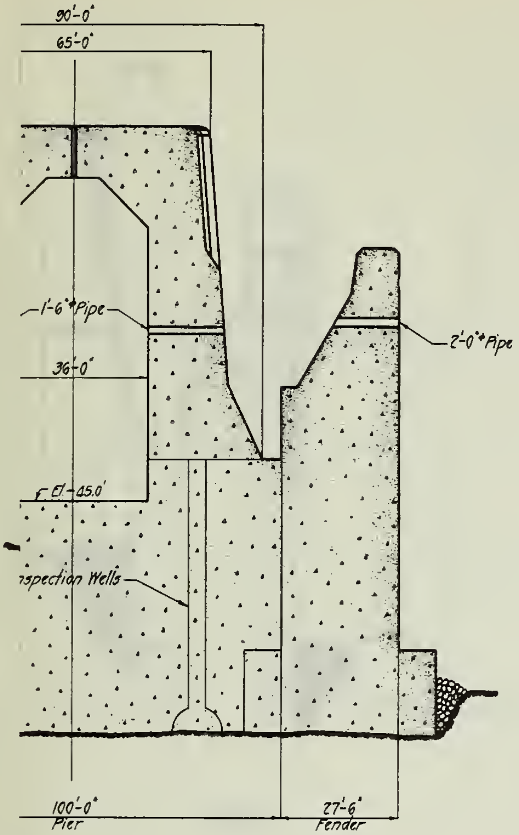
Westlake Palisades between bluff and highway in the foreground. Lake Merced district of San Francisco in left center. Park Merced Towers and Stonestown apartment buildings in center and left of center.

Photograph taken 9 months prior to the earthquake. (Moulin Studios photo, courtesy Theodore V. Tronoff.)

PART 7, SECTION D

DRAWINGS OF STRUCTURAL DESIGN OF THE GOLDEN GATE BRIDGE FROM THE REPORT OF THE CHIEF ENGINEER

(Refers to Various Sections of Part 2)



ꞮCTION 'B-B'

THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

SAN FRANCISCO PIER GENERAL PLAN AND SECTIONS

SCALE IN FEET

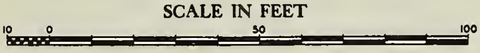
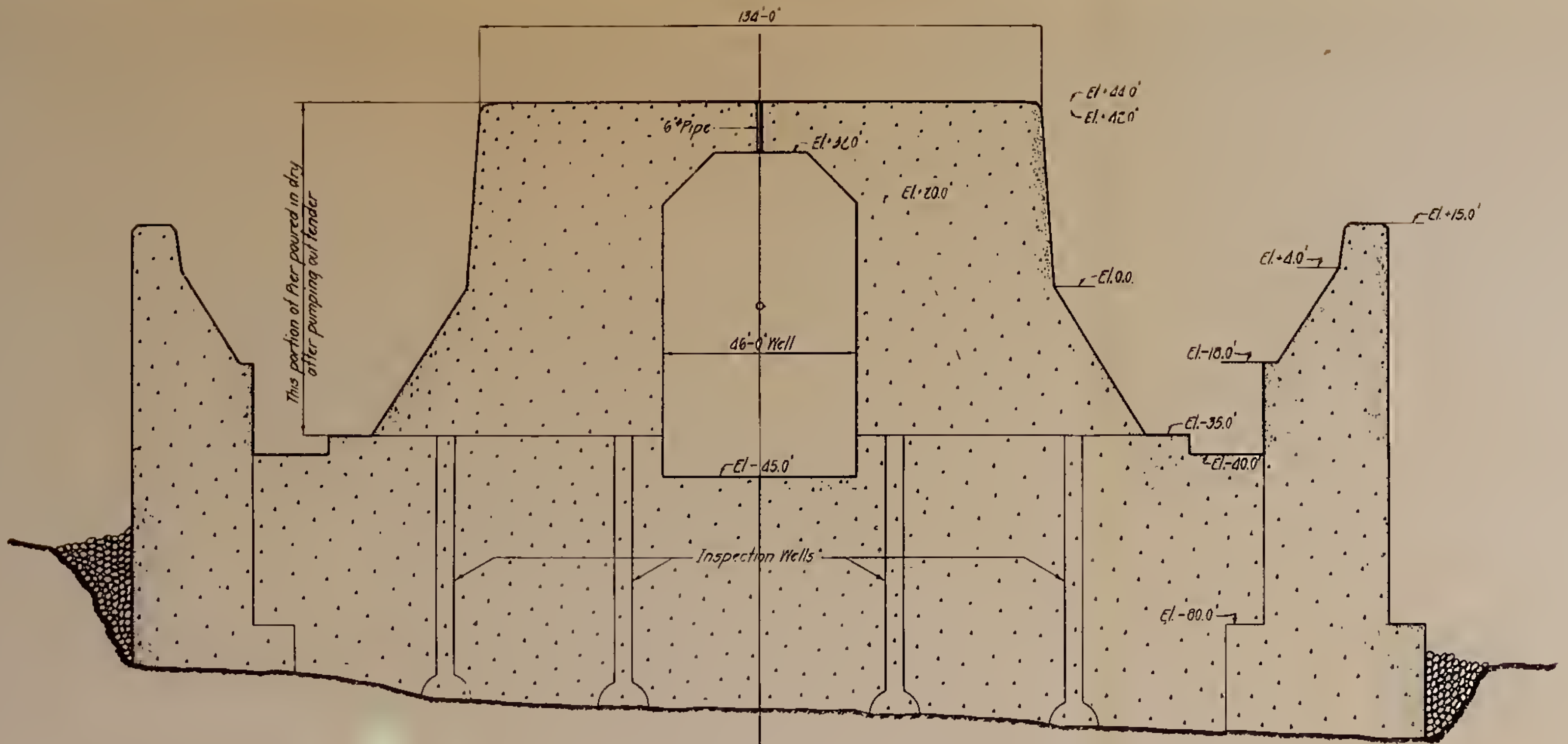
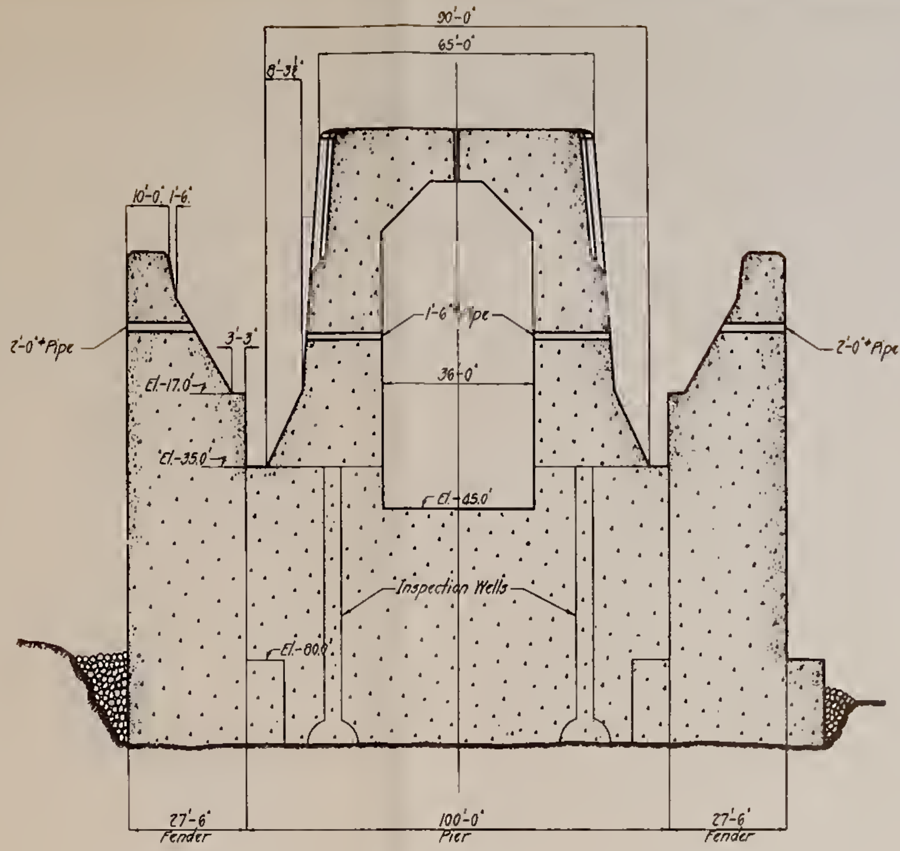


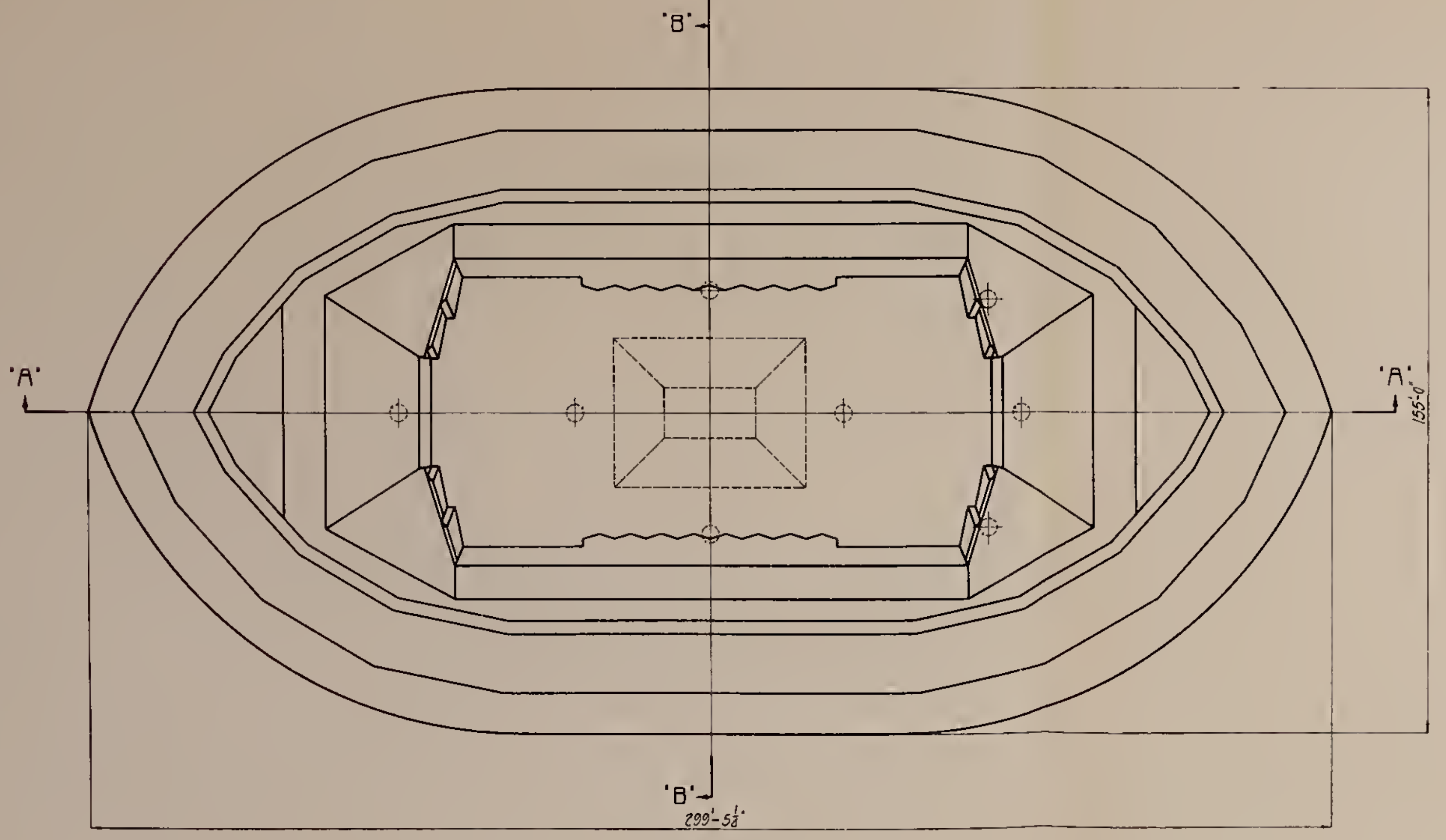
PLATE II



SECTION 'A-A'



SECTION 'B-B'



PLAN

NOTE:-
Tremie concrete for Fender to Elev. +5.0' and for Pier to Elev. -35.0'.
All other concrete poured in the dry.

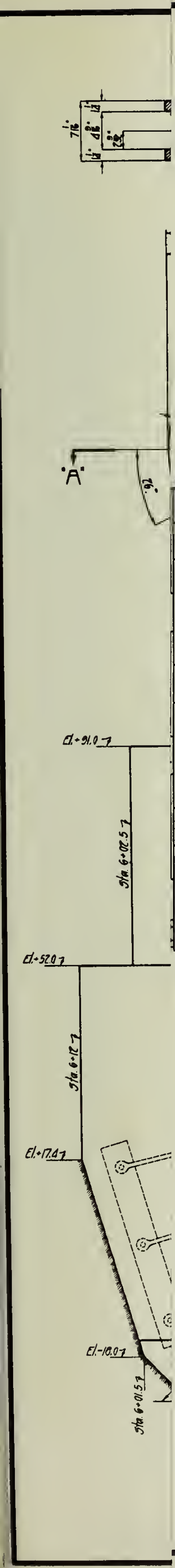
THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

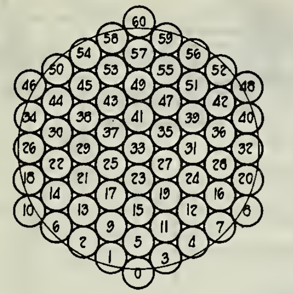
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

SAN FRANCISCO PIER
GENERAL PLAN AND SECTIONS

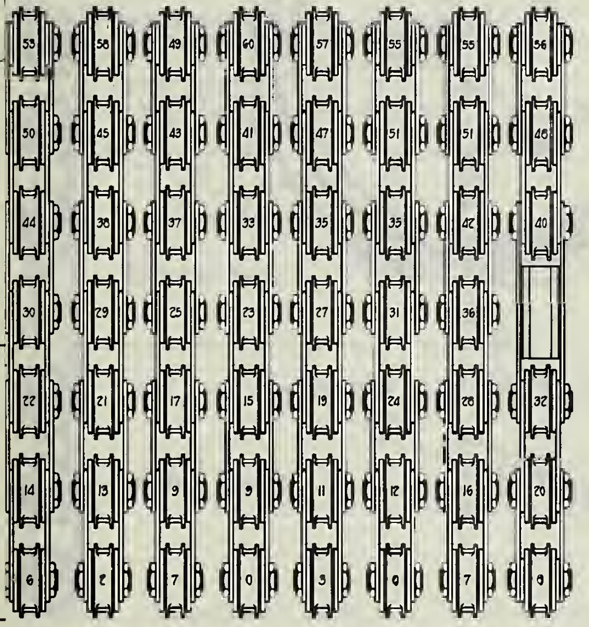
SCALE IN FEET

PLATE II

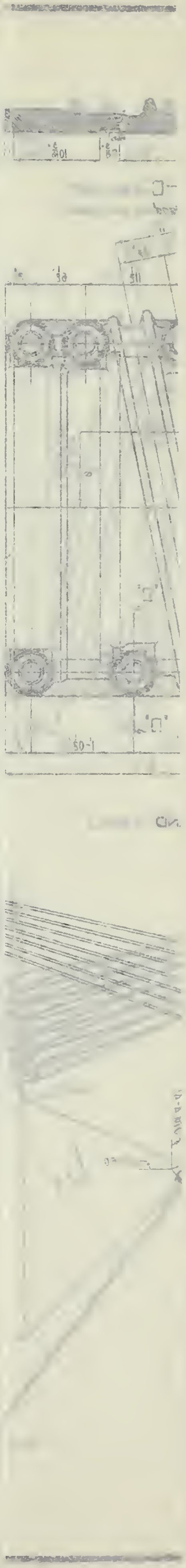




Cable Strands Before Compacting



Section 'E-E'
Showing location of Strand Shoes.



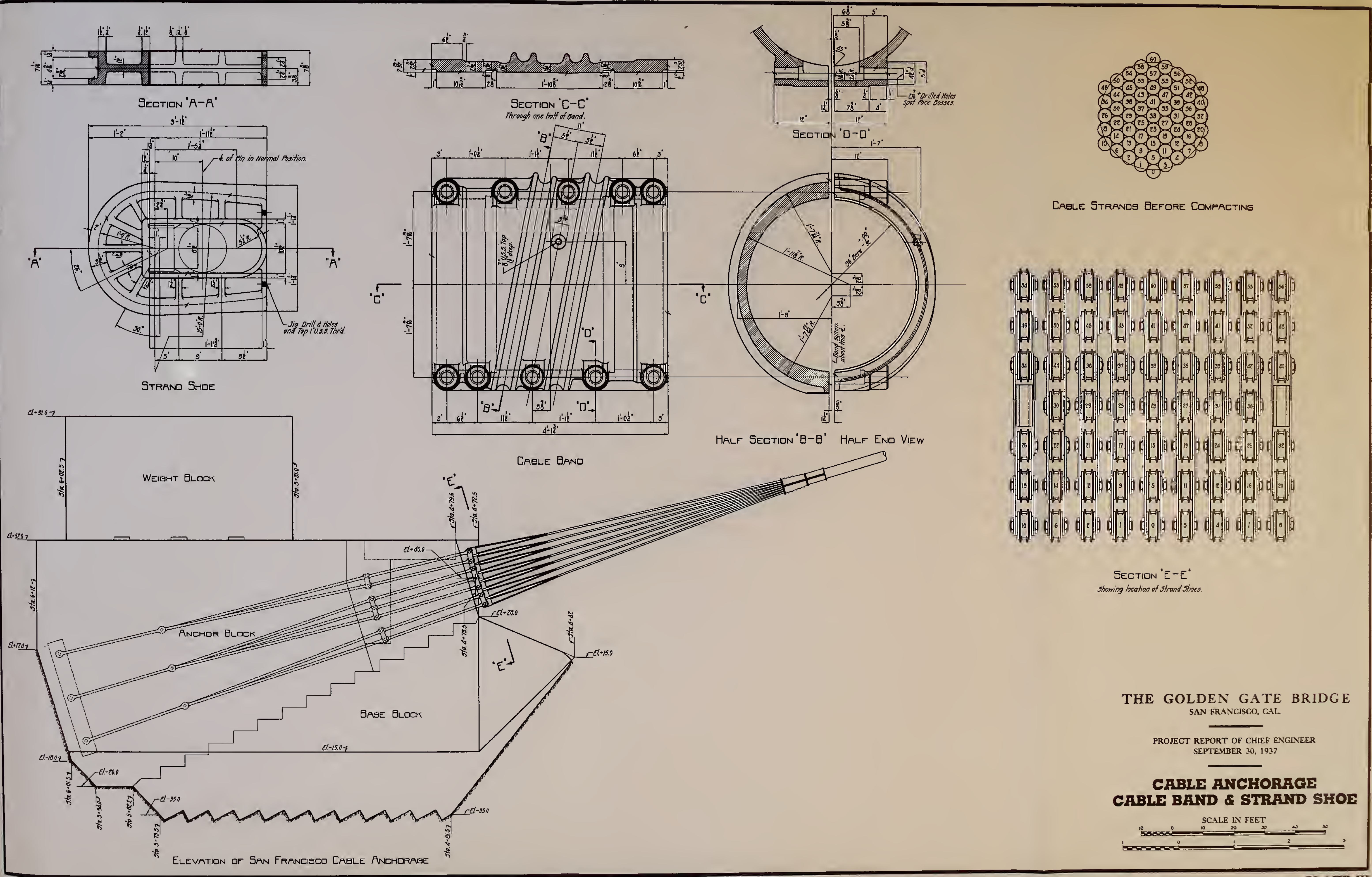
THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CABLE ANCHORAGE
CABLE BAND & STRAND SHOE

SCALE IN FEET

PLATE III


Section 'A-A'
Strand Shoe
℄ of Pin in Normal Position.
Jig Drill 4 Holes and Tap 1" U.S.S. Thr'd.
Section 'C-C'
Through one half of Band.
Cable Band
Section 'D-D'
Drilled Holes Spot Face Bosses.
Half Section 'B-B'
Half End View
Band symm. about this ℄.
Cable Strands Before Compacting
Section 'E-E'
Showing location of Strand Shoes.
Weight Block
Anchor Block
Base Block
Elevation of San Francisco Cable Anchorage

THE GOLDEN GATE BRIDGE

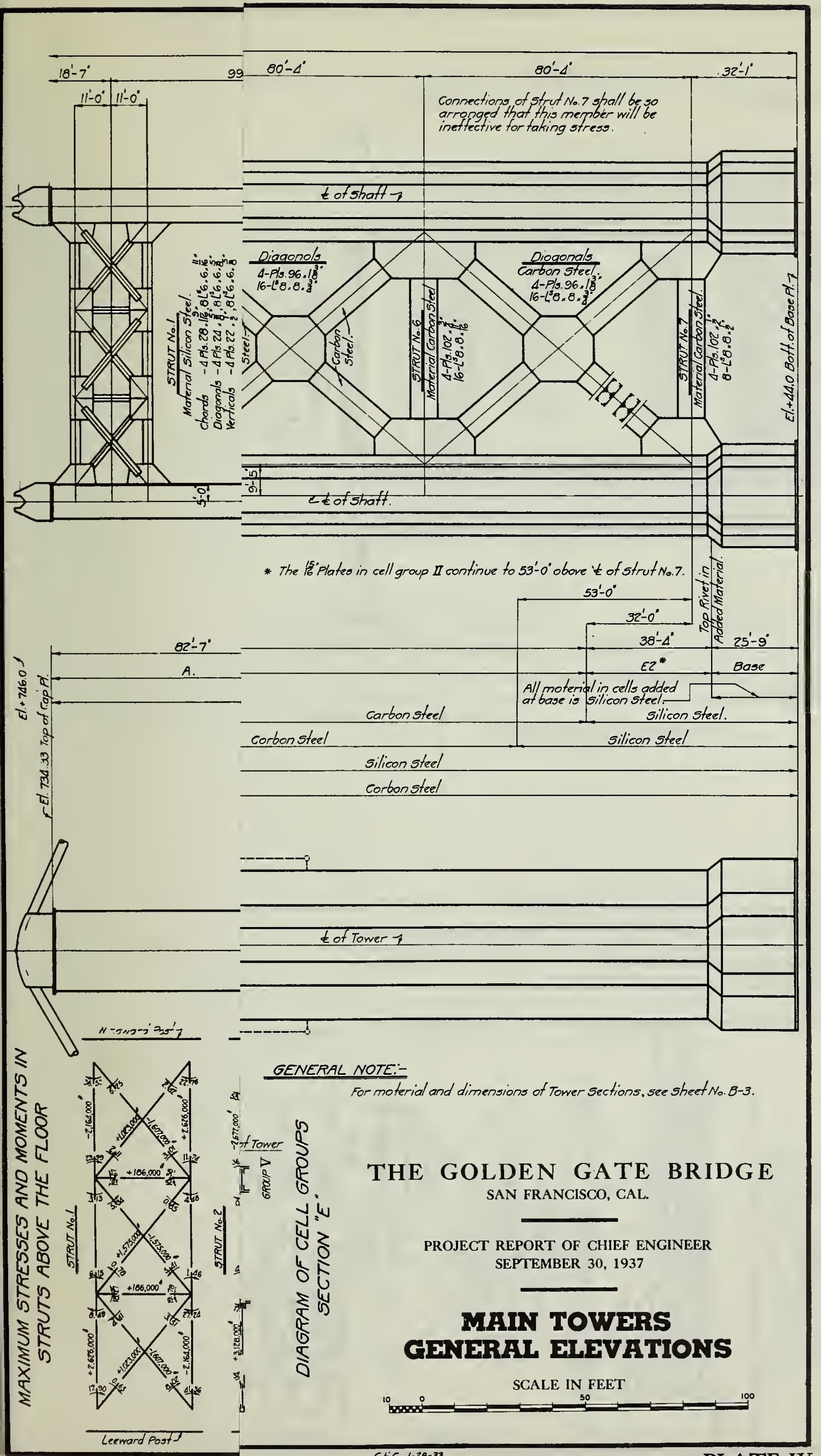
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CABLE ANCHORAGE
CABLE BAND & STRAND SHOE

SCALE IN FEET

PLATE III



THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

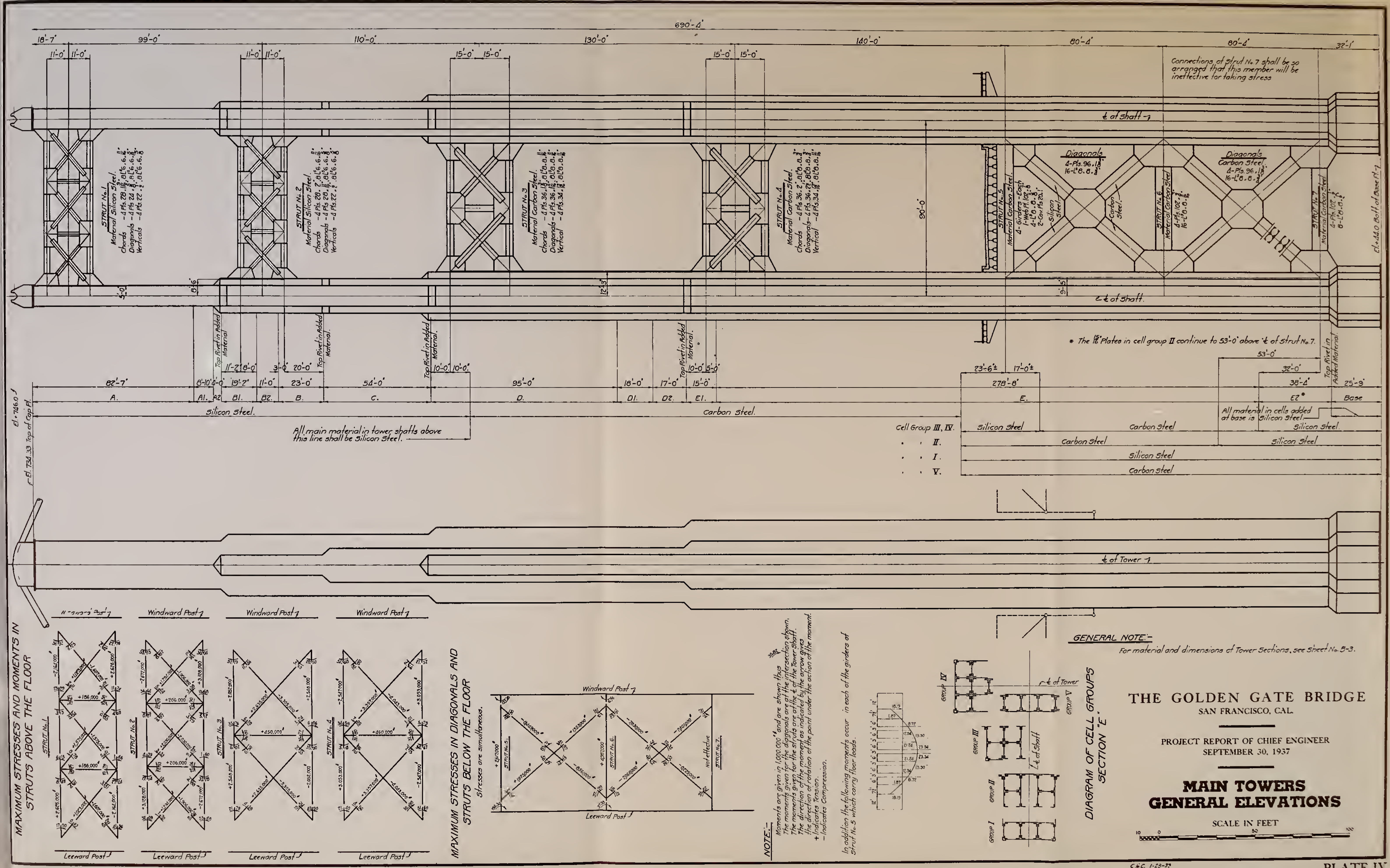
MAIN TOWERS GENERAL ELEVATIONS

SCALE IN FEET

10 0 50 100

C.H.C. 1-28-33
C.H.C. 2-7-33
A.H. 4-19-33

PLATE IV



Connections of Strut No. 7 shall be so arranged that this member will be ineffective for taking stress

* The 1 3/16" Plates in cell group II continue to 53'-0" above ℄ of Strut No. 7.

MAXIMUM STRESSES AND MOMENTS IN STRUTS ABOVE THE FLOOR

MAXIMUM STRESSES IN DIAGONALS AND STRUTS BELOW THE FLOOR

Stresses are simultaneous.

NOTE:-

Moments are given in 1,000,000"# and are shown thus 35.82. The moments given for the diagonals are at the intersection shown. The moments given for the struts are at the ℄ of the Tower Shaft. The direction of the moment as indicated by the arrow gives the direction of rotation of the point under the action of the moment.

+ Indicates Tension.

- Indicates Compression.

In addition the following moments occur in each of the girders of Strut No. 5 which carry floor loads.

DIAGRAM OF CELL GROUPS SECTION "E"

GENERAL NOTE:-

For material and dimensions of Tower Sections, see Sheet No. 5-3.

THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER

SEPTEMBER 30, 1937

MAIN TOWERS

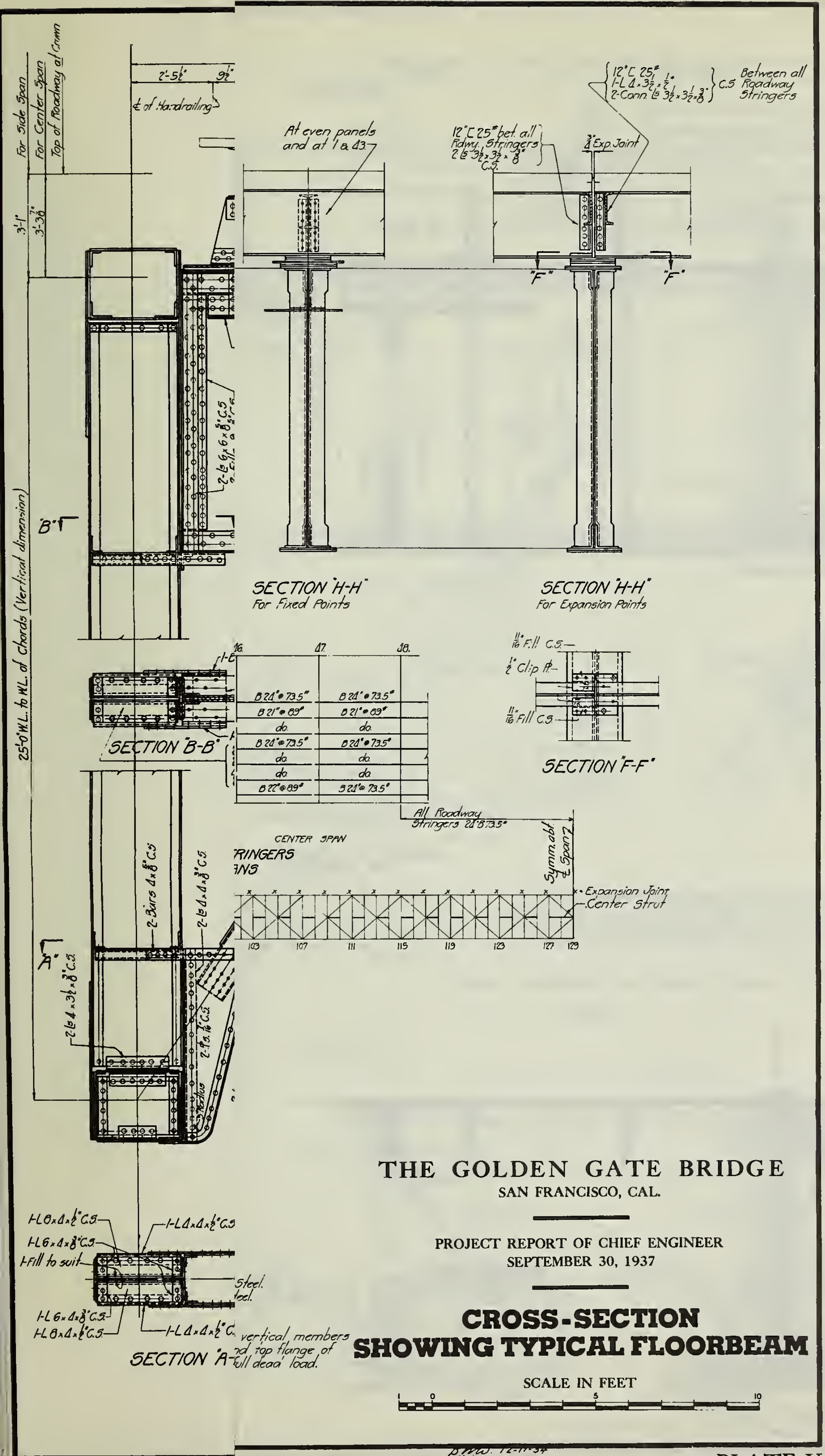
GENERAL ELEVATIONS

SCALE IN FEET

C.H.C. 1-28-33

C.H.C. 2-7-33

PLATE IV



THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

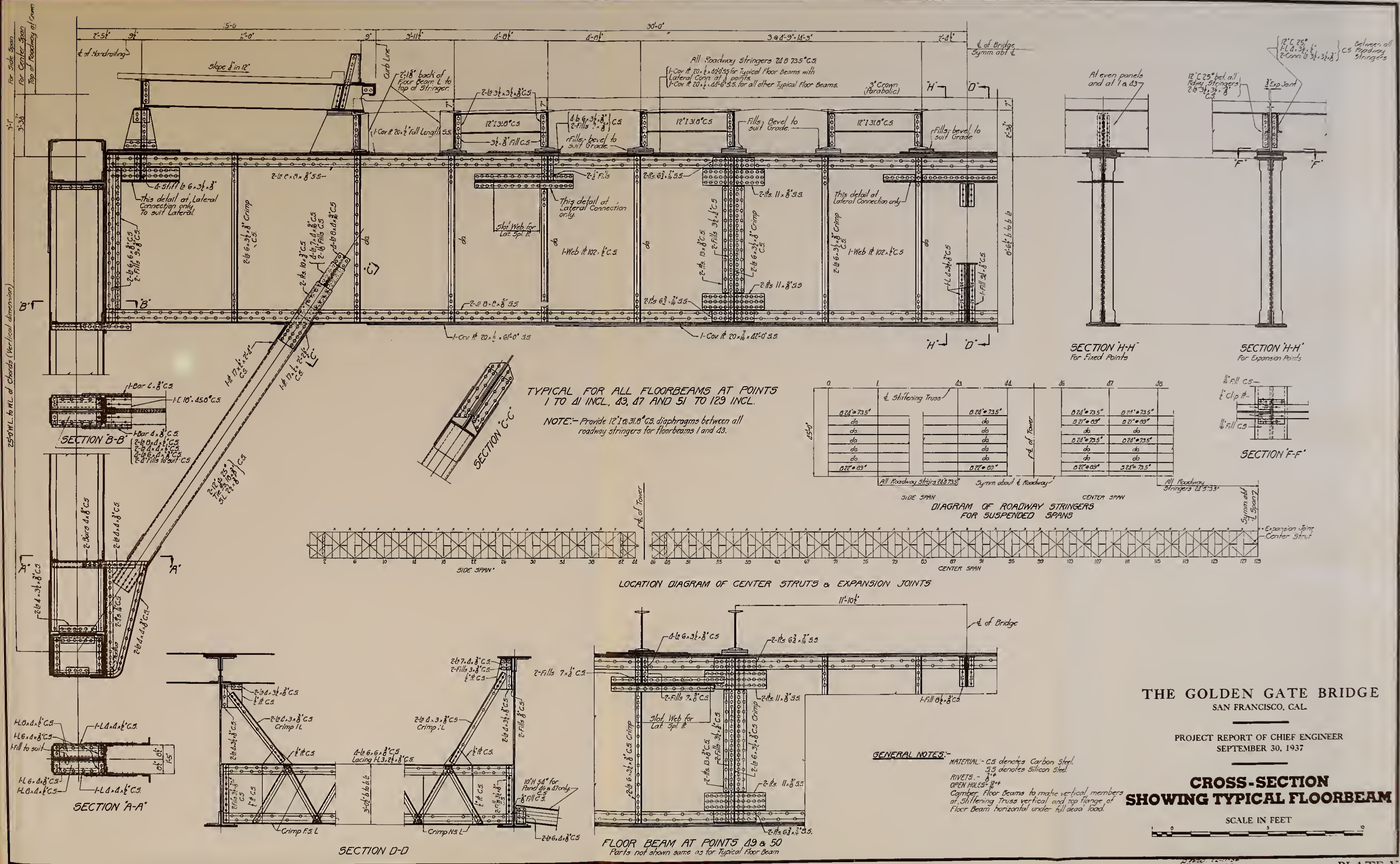
CROSS-SECTION SHOWING TYPICAL FLOORBEAM

SCALE IN FEET

0 5 10

D.M.W. 12-11-34
G.W.S. 1-5-35
Drwd. 1-30 35

PLATE V


For Side Span
For Center Span
Top of Roadway at ...
25'-0" W.L. to W.L. of Chords (Vertical dimension)
15'-0"
30'-0"
2'-5½"
1'-0"
3'-11½"
4'-8½"
3@4'-9"=14'-3"
2'-4½"
℄ of Handrailing
Slope ⅜" in 12"
Curb Line
℄ of Bridge Symm. abt ℄
All Roadway Stringers 24 B 73.5# C.S.
3" Crown (Parabolic)
12" I 31.8# C.S.
Fills; Bevel to suit Grade.
1-Web ℔ 102 x ½" C.S.
Crimp
This detail at Lateral Connection only. To suit Lateral.
This detail at Lateral Connection only.
Slot Web for Lat. Spl. ℔
SECTION "B-B"
SECTION "C-C"
SECTION "A-A"
SECTION "D-D"
SECTION "H-H" For Fixed Points
SECTION "H-H" For Expansion Points
SECTION "F-F"
At even panels and at 1 & 43
12" C 25# bet. all Rdwy. Stringers
⅜" Exp. Joint
TYPICAL FOR ALL FLOORBEAMS AT POINTS 1 TO 41 INCL., 43, 47 AND 51 TO 129 INCL.
NOTE:- Provide 12" I @ 31.8# C.S. diaphragms between all roadway stringers for floorbeams 1 and 43.
℄ Stiffening Truss
B 24" @ 73.5#
do
B 27" @ 83#
B 21" @ 83#
All Roadway Stringers 24 B 73.5#
Symm. about ℄ Roadway
All Roadway Stringers 24 B 73.5#
℄ of Tower
45'-0"
SIDE SPAN
CENTER SPAN
DIAGRAM OF ROADWAY STRINGERS FOR SUSPENDED SPANS
Symm. abt. ℄ Span
Expansion Joint
Center Strut
LOCATION DIAGRAM OF CENTER STRUTS & EXPANSION JOINTS
11'-10½"
FLOOR BEAM AT POINTS 49 & 50
Parts not shown same as for Typical Floor Beam

GENERAL NOTES:-
MATERIAL:- C.S. denotes Carbon Steel.
S.S. denotes Silicon Steel.
RIVETS.- ⅞"⌀
OPEN HOLES:- 15/16"⌀
Camber Floor Beams to make vertical members of Stiffening Truss vertical and top flange of Floor Beam horizontal under full dead load.

THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

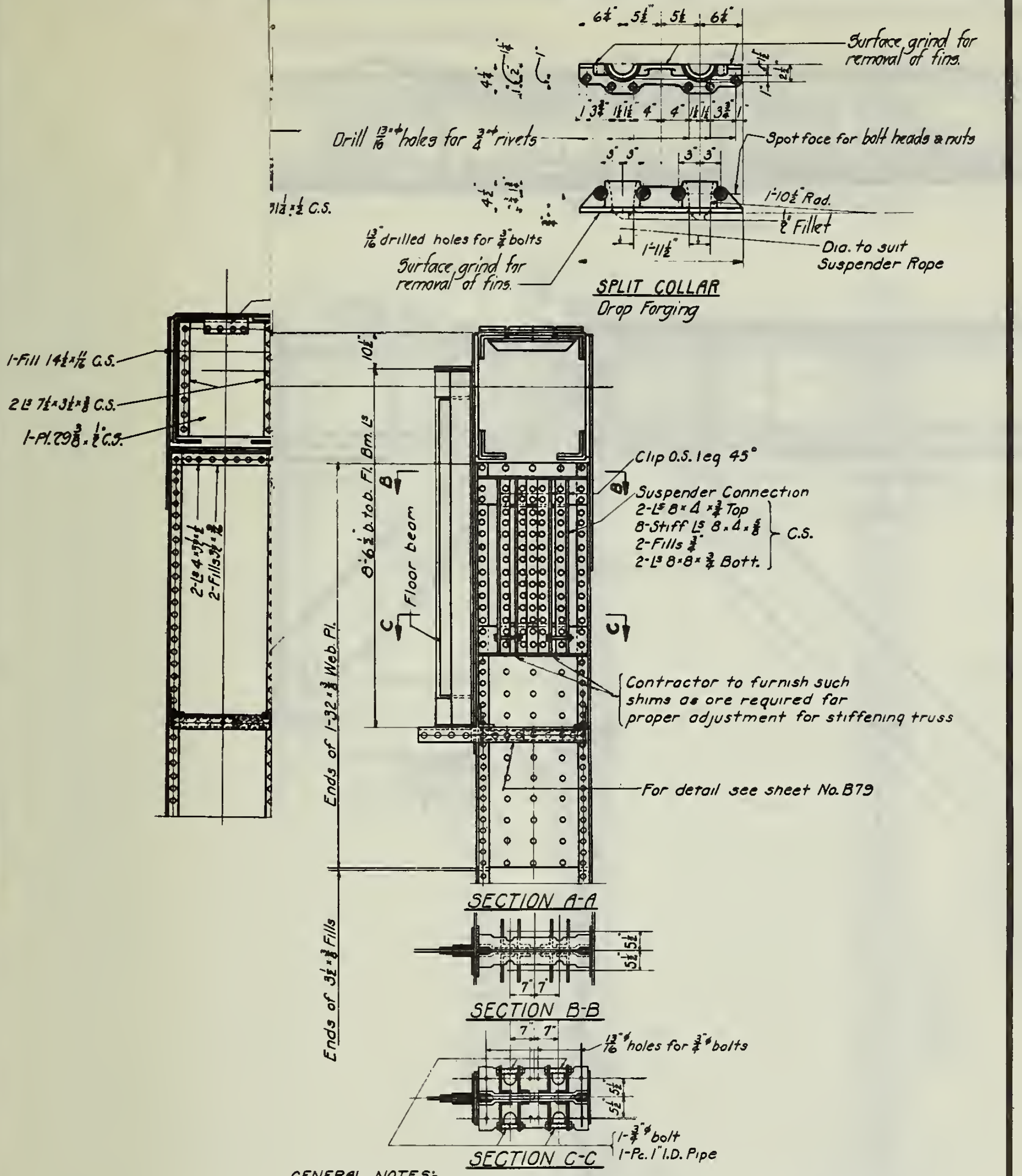
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

CROSS-SECTION SHOWING TYPICAL FLOORBEAM

SCALE IN FEET

1-5-35
1-30-35
3-2-36

PLATE V



GENERAL NOTES:-

For knee brace conn. to verticals and bottom chord see sheet No. B79

Truss to be detailed to make hangers and posts vertical under full dead load and normal temperature

Splices at U47 to U67 incl. same as shown for U53 except for fills

" " L48 to L68 incl. same as shown for L52

Material

S.S. denotes Silicon Steel

C.S. " Carbon Steel

Rivets 1" ϕ except ⅞" ϕ in 3½ legs of verticals and laced legs of diagonals

Open holes 1 1/16" ϕ except as noted

Maximum pitch of rivets not shown in chords and verticals to be 6"

THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

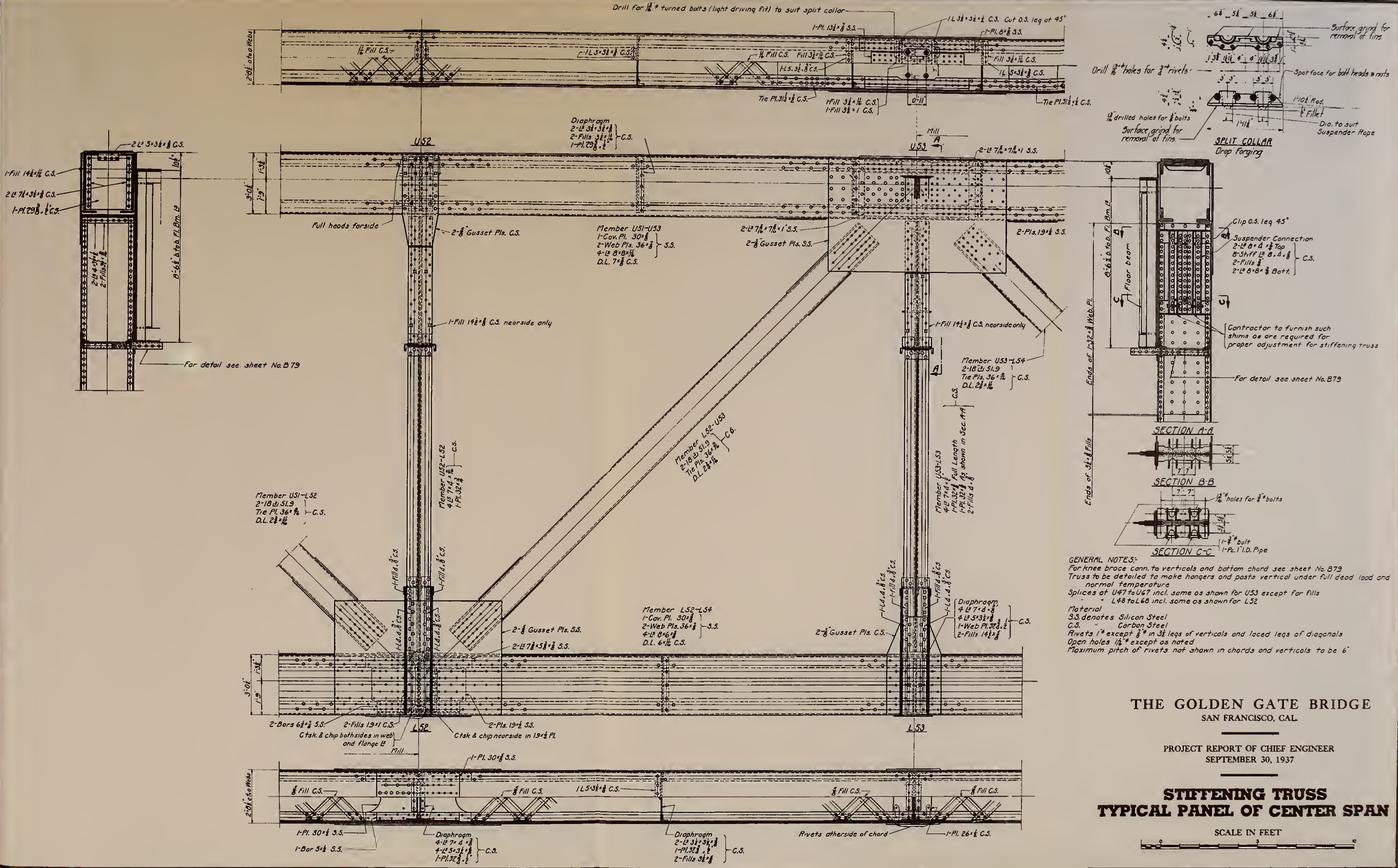
STIFFENING TRUSS
TYPICAL PANEL OF CENTER SPAN

SCALE IN FEET

0 5 10

12-11-34
1-5-35
1-30-35
3-21-35
35

PLATE VI



GENERAL NOTES:-
For knee brace conn. to verticals and bottom chord see sheet No. B79
Truss to be detailed to make hangers and posts vertical under full dead load and normal temperature
Splices at U47 to U67 incl. same as shown for U53 except for fills
" " L48 to L68 incl. same as shown for L52
Material
S.S. denotes Silicon Steel
C.S. " Carbon Steel
Rivets 1" except 7/8" in 3 1/2 legs of verticals and laced legs of diagonals
Open holes 1 1/16" except as noted
Maximum pitch of rivets not shown in chords and verticals to be 6"

THE GOLDEN GATE BRIDGE
SAN FRANCISCO, CAL.

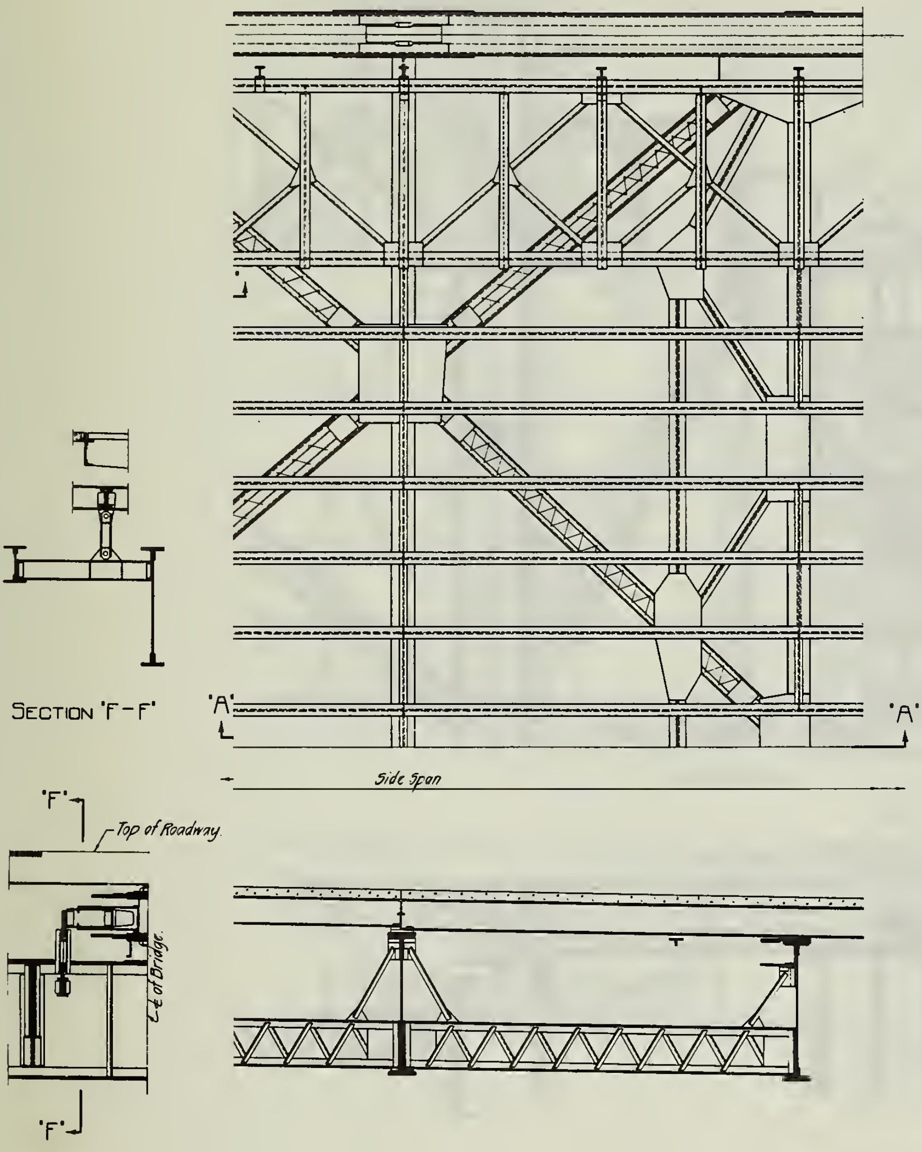
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

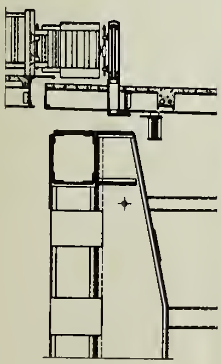
STIFFENING TRUSS
TYPICAL PANEL OF CENTER SPAN

SCALE IN FEET

1-5-35

PLATE VI





THE GOLDEN GATE BRIDGE

SAN FRANCISCO, CAL.

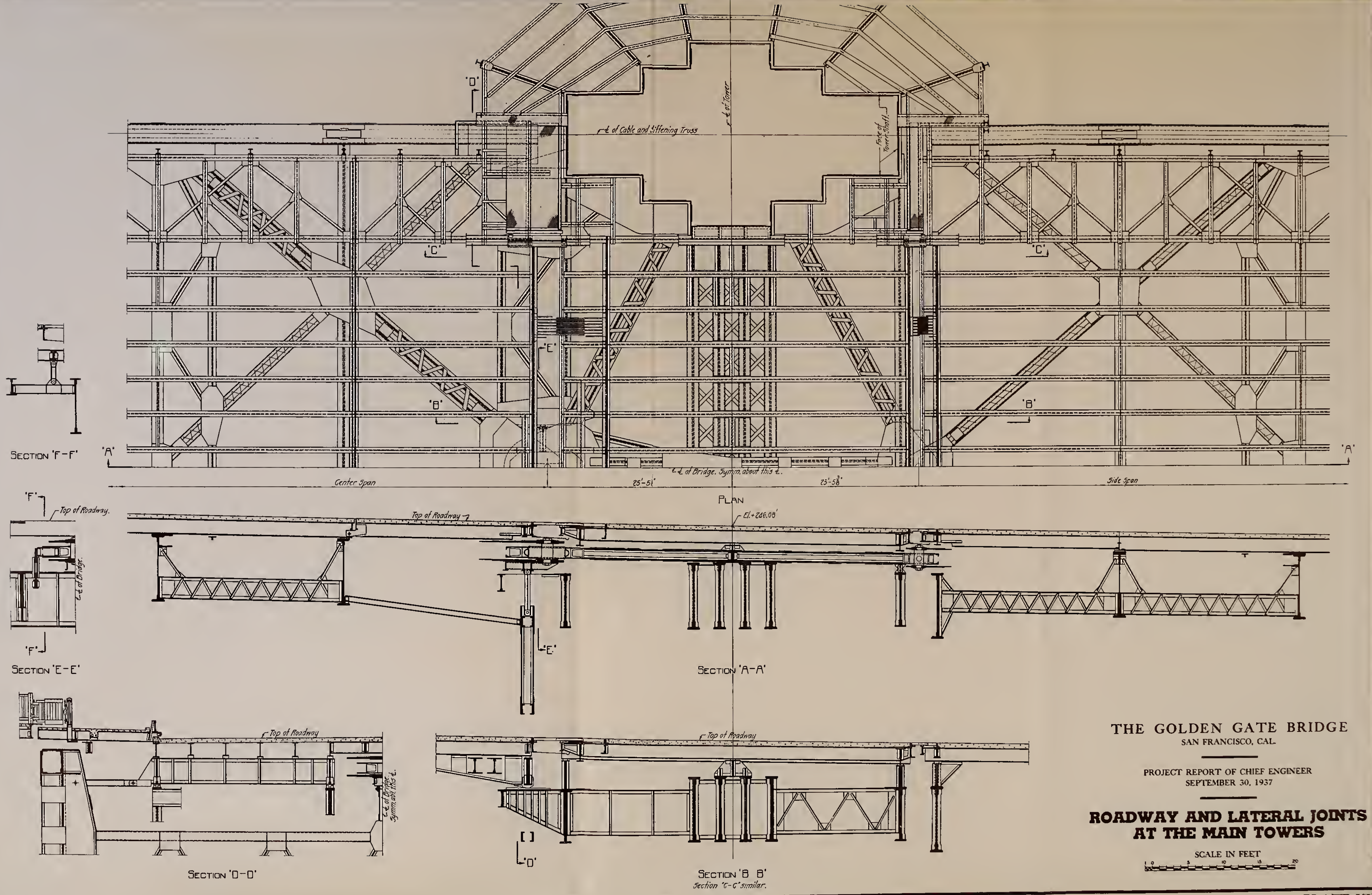
PROJECT REPORT OF CHIEF ENGINEER
SEPTEMBER 30, 1937

ROADWAY AND LATERAL JOINTS AT THE MAIN TOWERS

SCALE IN FEET

0 5 10 15 20

PLATE VII



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ROADWAY AND LATERAL JOINTS AT THE MAIN TOWERS

SCALE IN FEET

PLATE VII

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